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1960 Roster of Technical Committee Members on p. 2 of the News Letter

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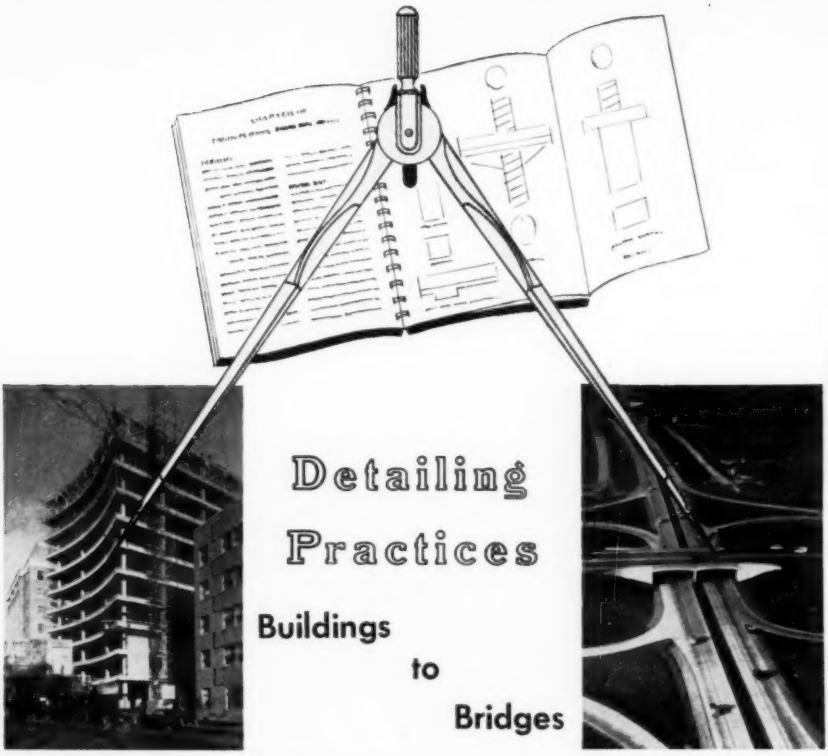

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HUBERT RUSCH opens the July JOURNAL with a report of work done by the Comité Européen du Béton on "Researches Toward a General Flexural Theory."

●

WEI-WEN YU and GEORGE WINTER present two simple methods for calculating instantaneous and long-time deflections under service loads in their paper, "Instantaneous and Long-Time Deflections of Reinforced Concrete Beams Under Working Loads."

●

The design and construction of the "Belgium Civil Engineering Pavilion," erected at the 1958 International Exhibition at Brussels, is described by A. PADUART and J. VAN DOOSSELAERE.

●

Coauthors JOHN E. BOWER and IVAN M. VIEST report on a series of tests sponsored by the Reinforced Concrete Research Council to determine the "Shear Strength of Restrained Concrete Beams Without Web Reinforcement."

●

LUIS P. SAENZ and IGNACIO MARTIN describe the design and construction of a series of reinforced concrete slab highway bridges in their paper, "Reinforced Concrete Slab Bridges for the Monumental Road, Havana, Cuba." The economics and the advantages of slab deck bridges for overhead crossings and underpasses are discussed.

On the Cover—The Philips Pavilion at the 1958 Brussels World Fair—composed entirely of shells having the form of hyperbolic paraboloids. For details see the feature article on p. 40 of the News Letter, "Prefabricated Prestressed 'Hypar' Forms Intricate Roof and Wall Structure."—Photo courtesy of Philips' Technical Review, N. V. Philips' Gloeilampenfabrieken, Eindhoven, Netherlands.

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Title No. 56-60

Elastic Analysis of Shear Walls in Tall Buildings

By EMILIO ROSENBLUETH and IGNACIO HOLTZ

Often the moment-resisting frame of a tall building is supplemented by shear walls running the full height of the building. Under lateral loads, interaction between the shear walls and frames, due to flexural deformations of the walls, leads to complex analytical problems. These are amenable to an approach by successive approximations similar to that of beams on elastic foundations. If the building is fairly symmetrical, a good first approximation can be obtained from the solution of a differential equation which is based on the assumption of complete uniformity. The solution to the differential equation is given as a set of graphs. The authors apply the method to one particular example. A numerical method is set up to systematize successive cycles of iteration and it is found that extrapolation from the first two cycles supplies almost exact results.

■ OFTEN IT IS ADVANTAGEOUS to resist lateral loads in tall buildings by a combination of moment-resisting frames and shear walls throughout the height of the building. This is particularly true in strengthening buildings damaged by earthquakes, for it is rarely practical to reinforce the majority of structural elements as would be required if the full lateral action were to be resisted by unaided frames. Sometimes drift limitations (which are desirable so as to avoid panic in buildings founded on soft soil, such as Mexico City clay), coupled with severe architectural limitations, also lead to the solution of combined shear walls and frames.

In "stocky" buildings, this type of structure does not as a rule call for elaborate analysis: the walls are usually much more rigid than the frames. Moreover it suffices to pay heed to shear deformations in the walls, and rough methods of analysis give ample accuracy.

The same type of structure in tall buildings has flexural deflections that can exceed ten times those due to shear. In turn, wall rotations induce important shears and bending moments in girders supported by them; indeed in the entire space frame. Analysis of interaction between shear walls and frames requires refined methods.

Emilio Rosenblueth is the director and a research professor of the Institute of Engineering, University of Mexico, Mexico D.F., Mexico. He has been associated with the university since 1951 and has also been active as a consulting engineer. An ACI member, he is active on ACI Committee 318 (Standard Building Code), ACI-ASCE Committee 328 (Limit Design), the ASCE committee on structural dynamics, and is active on Mexico's Federal District building code committee.

ACI member **Ignacio Holtz** recently joined D.C. Construcciones as a structural engineer. Prior to this he was employed at Diseno Racional, A.C. (DIRAC).

Usually the drift of a frame in any one story may be assumed proportional to the shear acting on the given frame in that story and independent of the shear in other stories within a wide range of shear distributions. Similarly one may assume that moments and shears in girders supported by a shear wall are proportional to the flexural slopes of the deflected wall. When this is the case the restriction offered by the rest of the structure to the shear-wall displacements consists of bending moments and shears proportional to the wall rotations. One can idealize each wall as a beam on an elastic foundation that differs from the classical Winkler foundation in that its responses are proportional to the wall rotations rather than to its displacements.

In general, a building possesses several shear walls in more than one direction. To every wall one may assign a certain number of tributary frames, or a tributary frame stiffness, and revise the assumption in subsequent cycles. For ease in presentation, however, it will be assumed that the building is not subjected to torsion and that it has not more than one shear wall in each direction, or that, from considerations of symmetry, this hypothesis is permissible. Such is the case in a symmetric building with symmetrically located shear walls. Admitting the presence of a single shear wall, the problem can be treated following the procedure of successive approximations described by Newmark,[1] adapted in a manner similar to that of Malter for beams on elastic foundations.[2]

The method requires assumption of a first configuration, which is improved in successive iteration cycles. When the wall is much more rigid than the rest of the structure, the assumption that the wall takes the entire lateral load supplies an acceptable first approximation. In other cases the initial estimate of the distribution of horizontal shear among wall and frames may differ widely from the solution and hence require an excessive number of iteration cycles to approach the answer. Accordingly, it is desirable to have a method supplying a rapid first approximation.

This paper develops two such methods. The first makes use of critical negative stiffnesses and is analogous to those proposed for buckling problems and forced vibrations.

The second method rests on the solution of a differential equation representing the behavior of a building idealized as having uniformly distributed mass and stiffness. Results, corresponding to a certain lateral load distribution, are tabulated in such a manner as to give a good guide for average but not excessively irregular buildings.

Notation

Symbols used in this paper are defined where they first appear with the most important ones listed below.

A = cross-sectional area of shear wall

C = base shear coefficient

E = modulus of elasticity

G = modulus of rigidity

H = height of building

h = story height

I = moment of inertia of shear wall cross section

K = shear stiffness (shear required to produce a unit change in slope of deflected structure)

$-k$ = ratio of actual to least critical stiffnesses

M = bending moment

$p^2 = \frac{H^2}{EI}\left(K_g + \frac{AGK_f}{AG + K_f}\right)$

$q = \frac{CH^2W}{EI\left(1 + \frac{K_f}{AG}\right)}$

r = extrapolation factor

V = shear

W = weight of building

x = coordinate measured downward from the top of the structure

$\xi = x/H$

ϕ = slope of deflected wall under action of lateral loads

Subscript f refers to frames, g to girders, m to effects of flexure, v to those of shear, and w to shear walls.

ITERATION PROCEDURE

Consider a shear wall forming part of a structure with moment resistant frames. The structure is subjected to lateral loads. Fig. 1 shows the deflected structure contained in the plane of the wall. Frames in other planes parallel to the wall undergo lateral displacements (in their own planes) practically identical with those of the wall, provided the

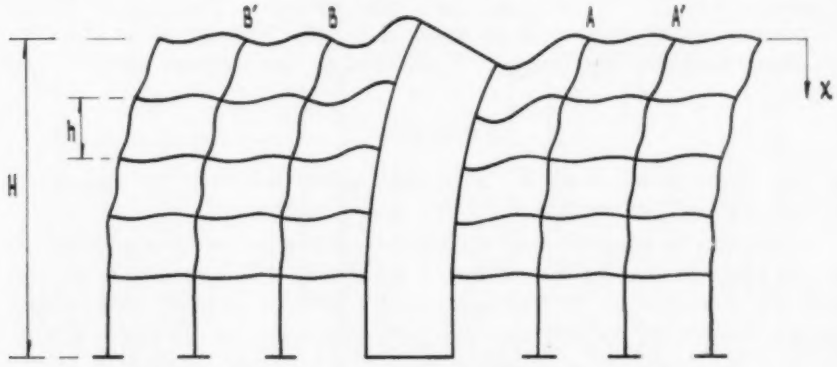


Fig. 1—Deflection of structure in plane of shear wall

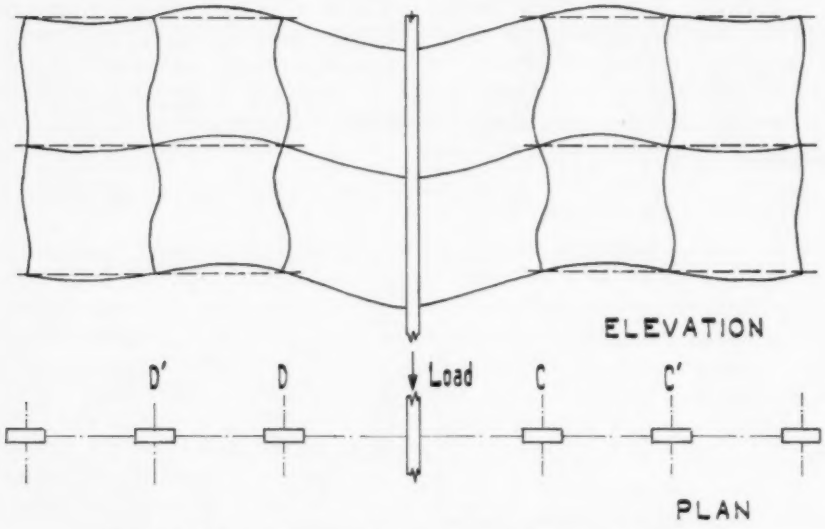


Fig. 2—Deformations of frames perpendicular to the wall

slabs are sufficiently rigid in their own planes. It will be assumed that this condition is fulfilled.

Frames contained in planes perpendicular to that in which the lateral loads act, and having girders supported by the shear wall, deflect as in Fig. 2.

It will be assumed that the shear taken by the frames in a given story is proportional to the average slope in that story, that is, to the ratio of the relative displacements of the floors under consideration over the story height, independently of the shear distribution. The ratio of shear to the average slope, ϕ, is therefore a constant for each frame in each story. It will be called the *frame stiffness*. The sum of frame stiffnesses in one story will be denoted by K_f. It is equal to the sum of shears taken by the frames, V_f, divided by the average slope of the story:

$$K_f = V_f/\phi \quad \text{.....(1)}$$

K_f can usually be obtained with reasonable accuracy by means of Wilbur's simplified formulas[3] or the factor method.[4]

It will also be assumed that the position of the points of contraflexure in the columns that lie closest to the shear wall (Columns A, B, C, and D in Fig. 1 and 2) can be estimated with sufficient accuracy and treated as independent of the shape of the deflected wall, say at midheight of the stories. A similar assumption will be made with respect to points of contraflexure in girders of spans adjacent to those which rest on the

wall, say at midspan (Spans AA′, BB′, CC′ and DD′). Accordingly, the moments, M_g, acting on the shear wall due to the deformations undergone by girders that rest thereon can be taken proportional to the flexural wall rotation, ϕ_m, assuming that the rotation due to shear, ϕ_v, does not produce moments in the girders. These moments are the sum of those acting at the wall support of girders contained in the plane of the wall, plus the sum of changes in reactions at the support of all girders, times the distances from the supports to the centroidal axis of the wall.

In principle, one should include torques induced in the slab and in girders perpendicular to the wall. However, computations for several specific instances show that the torques do not exceed a few percent of the sum of bending moments in the girders and can almost always be neglected.

The constant ratio of moments due to deformation of the girders, M_g, over the flexural rotation of the wall, divided by the height, h, of the story below the floor considered, will be denoted the *girder stiffness:*

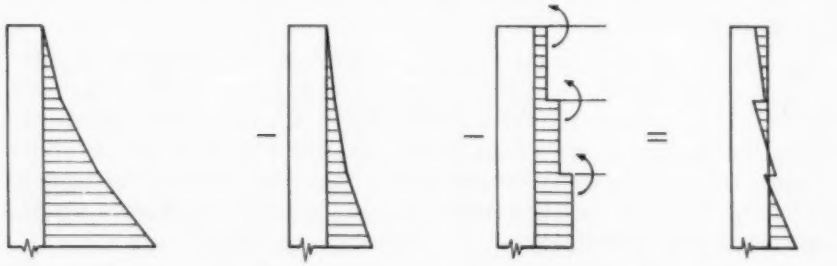
$$K_g = \frac{M_g}{h\phi m} \quad \text{(2)}$$

In each story the shear, V_w, taken by the wall equals the total shear, V, due to lateral forces minus that taken by the frames:

$$V_w = V - V_f \quad \text{(3)}$$

A similar expression can be set up for the overturning moments. However, a term must be included to represent the sum of moments taken by girders resting on the shear wall. The moment taken by the wall, M_w, is equal to the total moment, M, at the elevation considered minus that carried by the frame, M_f, and minus the sum of the moments taken by the girders, ΣM_g, from the top of the structure down to the elevation in question (Fig. 3):

$$M_w = M - M_f - \Sigma M_g \quad \text{(4)}$$



M = Total moment. M_f = Moment carried by frame. ΣM_g = Sum of girder moments. M_W = Moment in wall.

Fig. 3—Overturning moment diagrams

In Eq. (3) the shear in columns such as A, A', B, and B' in Fig. 1 has been neglected. These stresses are due to deformation of girders supported by the wall. Usually the effect is quite small but there would be no difficulty in taking it in consideration.

From the expressions which define the frame and girder stifness [Eq. (1) and (2)] one may obtain V_f, M_g, and, integrating V and V_f, one gets M and M_w. Substituting in Eq. (3) and (4)

$$V_w = V - K_f\phi \quad (5)$$

$$M_w = \int_0^x V_w dx' - \sum_{x=0}^{x} hK_g\phi_m \quad (6)$$

The integral and summation are performed with respect to coordinate x, measured downward from the roof of the building.

Once the moments and shears that act on the wall have been computed, the wall rotations can be obtained according to elementary strength of materials:

$$\phi_v = \frac{V_w}{AG} \quad (7)$$

$$\phi_m = -\int_H^x \frac{M_w}{EI}\, dx' \quad (8)$$

Here A represents the cross-sectional area of the wall, E and G its moduli of elasticity and rigidity, I its moment of inertia, and H the height of the building.

Actually Eq. (7) should be affected by a shape factor. Yet, experimental data by Coker[5] indicate that for the type of wall under consideration that factor is close to unity.

The sign convention adopted is such that, when the sum of lateral loads above a given section acts from left to right, the shear is positive. Compression in the right-hand fibers corresponds to positive bending moment; and slopes are positive when the deflected wall descends from left to right.

Eq. (5) to (8) supply the basis for a method of successive approximations. The method requires an initial estimation of the shear distribution, V_w, in the wall. From this, compute the bending moments M_w, and slopes ϕ_m and ϕ_v according to Eq. (7) and (8). Application of Eq. (5) and (6) leads to a new estimate of V_w and M_w. These constitute the beginning of a new iteration cycle, and so on, until one attains a sufficient degree of convergence.

In most slender walls, ϕ_v is negligible compared with ϕ_m. Computations can then be simplified, but the method as presented here is without this simplification.

The numerical procedure described will converge if the frames and girders are not too rigid when compared to the shear wall. The requirement is that stiffnesses K_f and K_g will not exceed certain critical values which, when the sign of the stiffnesses is changed, are capable of maintaining the wall deflected in a state of neutral equilibrium.

When the numerical method diverges, one can force convergence by resorting to an extrapolation procedure applicable to analogous problems solved by iteration. The procedure is due to Aitken[6] and has received a convenient form from Ryker.* It is presented below in Ryker's version.

Let X_0, X_1, and X_2 be, respectively, the assumed value, the result of the first cycle, and that of the second cycle, or the results of three successive cycles, of the shear taken by the wall, V_w, or of the slope, ϕ. (The X's represent generalized stresses or deformations in any problem solved by iteration.) Also let

$$\delta_1 = X_1 - X_0$$

$$\delta_2 = X_2 - X_1$$

$$r = \frac{\delta_1}{\delta_1 - \delta_2}$$

The extrapolated value of X is found from

$$X_{ex} = X_1 + r\delta_2 \qquad (9)$$

This expression is easily derived from the assumption that it is valid for all groups of three consecutive cycles.

Even when the iteration method converges, the extrapolation formula is useful because it accelerates convergence and therefore X_{ex} constitutes a better approximation than would be attained by carrying out several additional cycles.

In extremely bad cases of divergence it may become necessary to reapply Eq. (9), using for the X's three successively extrapolated values.

The magnitude of r indicates the manner in which the iteration process would converge or diverge if the extrapolation procedure were not resorted to. If $r \geqq 1$ the process converges monotonically; if $1 > r > 0.5$ it converges oscillating; if $0.5 \geqq r > 0$ it diverges oscillating; and if $0 \geqq r$ it diverges monotonically.

CRITICAL STIFFNESSES

Calculation of critical values of $-K_f$ and $-K_g$ may proceed by adapting Newmark's method for buckling loads.[1] The adaptation is so straightforward it does not merit a detailed exposition.

A variation proportional to the actual values must be allowed in all the stiffnesses, those denoted by K_f as well as those by K_g. Letting $-k$

*Based on personal communication from E. P. Popov, University of California, Berkeley, and N. M. Newmark class notes, University of Illinois, Urbana, 1950.

represent the (negative) ratio of actual stiffnesses to the fundamental critical values, slopes of the deflected shear wall should initially be taken as

$$\phi = \frac{\phi'}{1+k} \qquad (10)$$

where ϕ' represents the slopes of the deflected wall, assuming that the wall takes the entire load and is unaided by girders or frames.

Slopes computed from this formulas constitute a first approximation in the iteration process. The process converges even when the absolute values of actual stiffness exceeds the fundamental critical set, provided they are smaller than the second critical set. When they are smaller than the first set, Eq. (10) gives a first approximation leading to rapid convergence.

Actually, determination of critical stiffnesses requires practically as much computation as the complete solution of the problem. Hence, it is not justified unless the same structure is to be analyzed for a diversity of lateral load conditions. Many buildings are sufficiently uniform to permit a reasonable estimate based on the solution of a completely regular building; and this is easily obtained.

It will be assumed that the quantities AG, EI, K_f, and K_g do not vary from one story to the next and that the number of stories is sufficiently large to treat the second term in Eq. (6) as a continuous variable. Now, according to Eq. (5) and since $\phi = \phi_m + \phi_v$

$$V_w = V - K_f(\phi_m + \phi_v)$$

Substituting ϕ_v from Eq. (7) and transforming, we have

$$V_w = \frac{V - K_f\phi_m}{1 + \dfrac{K_f}{AG}} \qquad (11)$$

Substituting Eq. (11) in (6) and differentiating with respect to x

$$\frac{dM_w}{dx} = \frac{AG(V - K_f\phi_m)}{AG + K_f} - K_g\phi_m$$

and, from Eq. (8)

$$\frac{d^2\phi_m}{dx^2} = -\frac{1}{EI}\left(\frac{AGV}{AG+K_f} - K_g\phi_m - \frac{AGK_f}{AG+K_f}\,\phi_m\right)$$

or

$$\frac{d^2\phi_m}{d\xi^2} - p^2\phi_m = -\frac{AGH^2V}{EI(AG+K_f)} \qquad (12)$$

where

$$\xi = x/H$$

$$p^2 = \frac{H^2}{EI}\left(K_g + \frac{AGK_f}{AG+K_f}\right)$$

Parameter $-p^2$ will become critical when, with the total shear identically nil, the system admits a state of neutral equilibrium. Introducing the symbol

$$p'^2 = -p^2$$

and making $V = 0$, Eq. (12) becomes

$$\frac{d^2\phi_m}{d\xi^2} + p'^2\phi = 0 \qquad (13)$$

The general solution of this equation is

$$\phi_m = a_1 \sin p'\xi + a_2 \cos p'\xi$$

where a_1 and a_2 represent arbitrary constants.

Boundary conditions are

$$\left.\frac{d\phi_m}{d\xi}\right|_0 = 0, \qquad \phi_m(\xi = 1) = 0 \qquad (14)$$

Hence, $a_1 = 0$ and

$$p' = \frac{2n-1}{2}\pi$$

where n is any positive integer. The least critical value is obtained when $n = 1$. Therefore $p'_{cr} = \pi/2$. Accordingly $p^2_{cr} = -(\pi/2)^2$, and the least critical combination of stiffnesses satisfies the relation

$$\left(K_s + \frac{AGK_f}{AG + K_f}\right)_{cr} = -\left(\frac{\pi}{2H}\right)^2 EI$$

Redefining $-k$ accordingly

$$k = -(p/p_{cr})^2 = (2p/\pi)^2$$

and using average values for the stiffnesses in question, Eq. (10) becomes

$$\phi = \frac{\phi'}{1 + (2p/\pi)^2} \qquad (15)$$

If the stiffnesses vary in an exceedingly pronounced manner throughout the height of the building it may be worth idealizing their variation and setting a differential equation analogous to Eq. (13) but with a variable coefficient p'.

NONHOMOGENEOUS DIFFERENTIAL EQUATION

When the second member in Eq. (12) can be approximated by a polynomial in ξ, it is then a simple matter to solve the nonhomogeneous equation. The initial estimate of the deflected wall slopes is not necessarily proportional to ϕ', as was the case in Eq. (10). Better results can then be anticipated.

For example, several building codes require, in aseismic design, the assumption of a linear variation of horizontal accelerations, from zero

at the base of the structure to a maximum at the top. If the mass of the building is assumed uniformly distributed throughout its height, the total shear will vary according to the second-degree parabola

$$V = CW(2\xi - \xi^2) \quad (16)$$

where C is the base shear coefficient and W the weight of the building.

With the notation

$$q = \frac{CH^2W}{EI\left(1 + \frac{K_f}{AG}\right)}$$

Eq. (12) can be put in the form

$$\frac{d^2\phi_m}{d\xi^2} - p\phi_m = -q\ (2\xi - \xi^2) \quad (17)$$

Corresponding to the boundary conditions in Eq. (14), the solution of this differential equation is

$$\frac{p^2\phi_m}{q} = \left(1 - \frac{2}{p^2}\right)\left(1 - \frac{\cosh\ p\xi}{\cosh\ p}\right) + \frac{2\ \sinh\ p(1-\xi)}{p\ \cosh\ p} - (1-\xi)^2 \quad (18)$$

In Fig. 4 appear values of ϕ_m/q as a function of ξ and p^2. They were computed from this expression. In specific problems, slopes due to flexure, which can be obtained in Fig. 4, may be substituted in Eq. (11) in conjunction with actual (not average) values of K_f and AG, to obtain the shear taken by the wall in each story. To this first approximation there should follow a series of iterative trials.

In the present exposition overturning moments have not been reduced, mainly because the corresponding practice is not unified. One

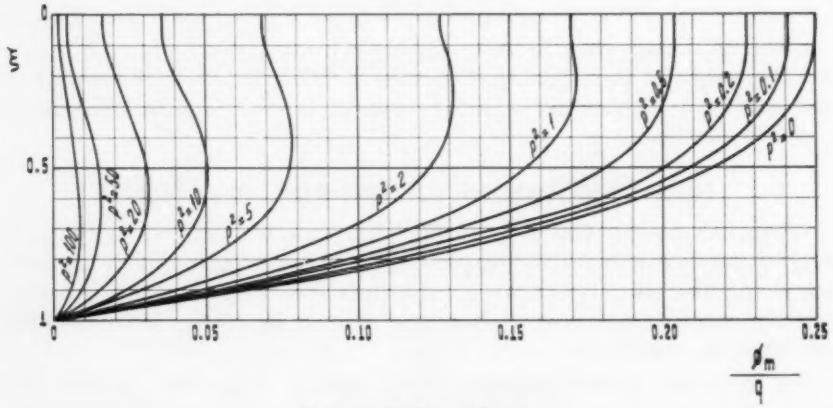


Fig. 4—Values of ϕ_m/q

simple manner of taking into consideration reductions in overturning moment consists in the use of modified values of EI.

EXAMPLE

Some of the methods described in this paper are illustrated by applying them to the structure depicted in Fig. 5. It corresponds to a nine-story building having a shear wall that runs the full height of the structure. It is to be analyzed for seismic forces acting parallel to the wall. A triangular distribution of horizontal acceleration is specified, with a base shear coefficient of 0.05. No reduction in overturning moment is allowed. Fig. 5 includes other pertinent data.

Frame stiffnesses were computed by using Wilbur's formulas and are tabulated in Table 1. Also tabulated are the girder stiffnesses, K_g, computed as described in the text; the weight assumed concentrated at each floor, and the total shear, V.

The average frame stiffness, $\overline{K}_f$, is 50,664 tons according to the table. The average girder stiffness, $\overline{K}_g$, is 25,956 tons. Therefore

$$1 + \frac{K_f}{AG} = 1.036$$

$$p^2 = \frac{27.50^2}{18.52 \times 10^6}\left(25{,}956 + \frac{50{,}664}{1.036}\right) \qquad q = \frac{0.05 \times 27.50^2 \times 3455}{18.52 \times 10^6 \times 1.036}$$

$$p^2 = 3.05 \qquad q = 0.00685$$

In every floor the mean slope due to flexure is found by using Fig. 4. The results are listed under ϕ_{mo} in Table 2. The shears V_{wo} in the table were computed from Eq. (11) by substituting in that expression the stiffness K_f of each story. These shears should be compared with the values V'_{wo}, which were computed from Eq. (10) and (11) with

$$k = (2p/\pi)^2$$
$$k = 3.78$$

whence

$$\phi = \frac{\phi'}{1 + 3.78}$$

Moments taken by the girders resulted from multiplication of slopes due to flexure, ϕ_m, by the stiffnesses hK_g.

Beginning with V_{wo} and M_{go}, slopes due to shear were obtained, as well as bending moments and new slopes due to wall flexure.

TABLE 1 — FRAME STIFFNESSES AND SHEAR

Floor	K_f, ton	K_g, ton	Weight, ton	V, ton
9		22,600	275	
	31,220			26.1
8		24,500	350	
	37,580			55.7
7		25,200	350	
	44,280			81.6
6		26,400	400	
	47,570			107.2
5		26,600	400	
	50,800			128.6
4		27,400	400	
	56,380			145.9
3		28,000	400	
	64,850			159.0
2		28,400	400	
	64,380			167.9
1		24,000	400	
	58,620			172.7
0			80	

From the frame and girder stiffnesses new values of V_g and M_g were derived, and hence a new set of shears taken by the wall (V_{w1} in Table 2) and girder moments (M_{g1}). In view of the results found, a second cycle was deemed unnecessary.

By comparing V'_{wo} with V_{wo}, it is seen that the latter shears lie much closer to the answer.

The same problem was reanalyzed beginning with arbitrary values of the slopes ϕ. After two cycles, results were as far from the answer

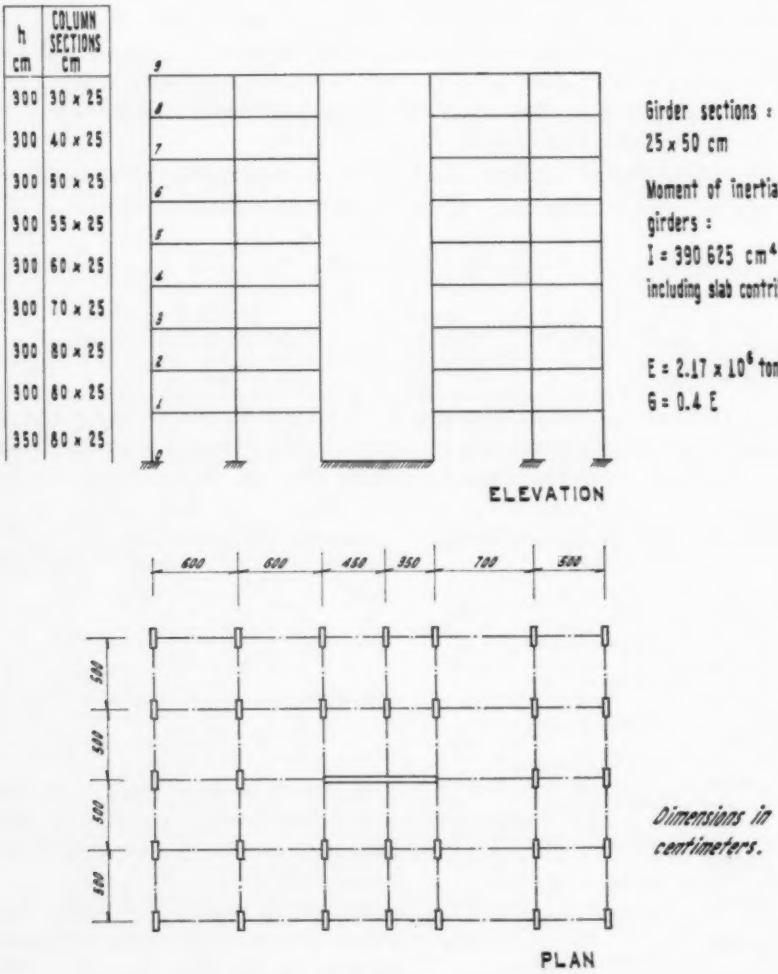


Fig. 5—Application example

TABLE 2 — SLOPE DUE TO FLEXURE, MOMENT, AND SHEAR

Floor	ϕ_{mo}	V_{wo}, ton	$\frac{\phi'}{1+k}$	V'_{wo} ton	M_{go}, ton	V_{wt}, ton	M_{gt}, ton
9					49.5		49.0
	0.000735	3.0	0.000800	1.1		3.4	
8					54.0		54.0
	0.000750	26.6	0.000792	25.7		27.0	
7					57.1		56.5
	0.000765	46.2	0.000773	47.4		46.9	
6					60.5		59.0
	0.000757	69.0	0.000738	72.2		69.2	
5					59.2		59.0
	0.000720	89.2	0.000676	94.4		89.4	
4					57.0		57.0
	0.000655	105.0	0.000592	112.3		105.5	
3					52.0		51.5
	0.000557	117.0	0.000470	128.6		119.5	
2					41.5		41.0
	0.000402	134.5	0.000319	147.4		138.0	
1					25.0		25.3
	0.000162	156.0	0.000127	165.3		158.0	

as the slopes assumed. Yet by applying the extrapolation procedure at this stage, almost exact results are obtained.

A more complete analysis would require a moment distribution in the frames, assuming that they deflect when the walls do. This would lead to a revision of effective stiffnesses and perhaps additional trials so as to improve the accuracy.

CONCLUSION

The problem of interaction between shear walls and moment resistance frames in tall buildings can be dealt with by a numerical procedure of successive approximations. The procedure is analogous to those used for beams on elastic foundations. Rapid convergence is achieved if the initial approximation constitutes the solution of a certain differential equation.

In the case of arbitrary lateral load distributiton, deflections of the wall can advantageously be assumed proportional to those which it would undergo if it took the full lateral load, deducing the constant of proportionality from the minimum critical stiffness of a uniform building.

If the shear distribution can be closely approximated by a simple polynomial, the solution of the nonhomogeneous equation will give a better estimate of the amount of shear the walls will withstand. The charts in this paper are useful for the special case of parabolic distribution of total shears.

In any case, convergence is greatly accelerated by resorting to the extrapolation procedure described in the paper.

The assumption of end fixity at the base of the wall can be dispensed with. It is sufficient to introduce the appropriate boundary condition

at $\xi = 1$, and solve the differential equations accordingly. For example, foundation and soil conditions justify the assumption that the flexural rotation, ϕ_m, at $\xi = 1$, is proportional to M_w at that elevation. However, coverage of such general conditions in the form of charts would require an excessive number of these charts, making their use impractical. When it is intended to cover conditions other than fixity or a hinge at the base of the shear wall, it will be found preferable to use the successive approximations method aided by extrapolation.

Lest the mathematical tone of this paper give the impression that extreme accuracy is expected, it is well to point out that the present knowledge of shear walls is as yet quite obscure. Regardless of the method of analysis, grave uncertainties remain concerning actual stresses and safety of the structure. These doubts must invariably be offset by a judicious choice of load factors and working stresses.

Under the circumstances, extreme refinement in analysis is not justified. Still, the methods described do seem to offer means to simplify computations, whatever the degree of accuracy desired.

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Method of Assessing Probable Fire Endurance of Load-Bearing Columns

By J. H. CLARKE

This paper gives two methods of assessing the probable fire endurance of load-bearing reinforced concrete columns subjected to the standard fire resistance test. A column during a test may support a load well below the permissible design load and therefore the fire endurance period may be expected to exceed the fire resistance rating allocated according to the size of the column, thickness of the cover, reinforcement, and the load.

The suggestion is tentatively put forward that designers may be permitted to design a column according to the load carried, including a safety factor. Then they may modify the design by the methods given in this paper if the requisite fire resistance is not obtained.

The column sizes dealt with are 15 x 15-in. to 20 x 20-in. as this range of column sizes occurs frequently in design practice and information on the smaller sizes is more readily obtainable.

THE ACTUAL TIME DURING which a reinforced concrete column can support a load, while being subjected to the standard fire resistance test, is termed the fire endurance of the column. The conditions of the test are approximately the same in the various countries where these tests are conducted, but there are some important differences.

The design load for a given column varies according to the standard adopted by the country conducting the test. In some cases, the column is subjected to a stream of water from a fire hose at the end of a stipulated test period. The load may be removed from a column and returned after a lapse of 2 days. If luminous furnace gases are employed, a less uniform temperature distribution will occur throughout the furnace. The thermocouples surrounded by the hot gases may, or may not, be shielded to reduce radiation from the hot junctions to the cooler parts of the furnace or the specimen. This condition will effect the severity of the exposure of the specimen to the furnace. It is not a

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simple problem to distribute an applied load uniformly over the cross-sectional area of the column and any appreciable discrepancy here will effect the mode of failure of the column.

Hence it will be seen that before a fire resistance rating is assigned to a load-bearing reinforced concrete column, expert interpretation of the conditions must be obtained. In addition to the conditions already mentioned, it is obvious that some account must be taken of the nature of the aggregate employed, the amount of free moisture present during the test, the depth of the cover to the reinforcement, and any additional protection to the reinforcement, e.g., the employment of light mesh to reduce spalling. Finally, the length of the curing period and the actual curing conditions must be examined to see if the standard conditions are met.

Calculations can be made of the temperatures within columns for any time interval of a fire resistance test, but it is necessary to know the values of the thermal constants to within 15 percent if the results are to be of any use. The presence of the reinforcement adds to the difficulty by increasing the thermal diffusivity locally. This accuracy is not readily obtained unless the nature of the concrete is fully known. The unknown quantity of free water present in the concrete at the time of the test is a further source of error. It is, however, possible to estimate the maximum temperature at any point during a test, and if this maximum temperature is known then the maximum effective compressive strength of the concrete comprising the column can readily be found.

The results and details of fire resistance tests on reinforced concrete columns, carried out in the United States and England, are given in References 1, 2, and 3.

The fire resistance test, by subjecting the columns to a fast rising temperature, increases the likelihood of spalling above what would be expected from the majority of actual building fires. In fact, examination of the after effects of a fire rarely discloses extensive spalling. Furthermore, it is believed that a census of building fires might show that most buildings which have been subjected to fire have been in service for a number of years. Hence the concrete has had time to reach an air-dry condition and is less likely to spall. Testing laboratories are seldom able to wait more than 1 year, and therefore some allowance should be made for spalling, if it occurs. Again, where ade-

quate fire fighting services exist in close proximity to a building, some relaxation in the fire ratings of the elements of construction could perhaps be tolerated, especially if an efficient sprinkler system has been installed.

With these facts in mind, some legislating authorities are considering permitting the use of main structural elements in certain public buildings with fire ratings less than 4 hr. Thus the methods advocated in this paper may, it is believed, present a new and adequate approach to dealing with the fire resistance requirements of reinforced concrete columns.

CONSTANTS EMPLOYED IN DETERMINING THE LOAD-BEARING CAPACITY OF CONCRETE

Relationship (1) — A 6-in. concrete cube has 1.12 times the compressive strength of a cylinder 6-in. in diameter and 12 in. long, provided that both have been cured by the standard method for 1 year (see Reference 4, Table VI).

Relationship (2) — Kesler has shown that an alternative relationship might be:

$$\text{cylinder strength} = 0.92 \times \text{cube strength} - 200$$

all compressive strengths being in psi. This relationship is for practical purposes sufficiently close to the previous relationship (see Reference 4, Fig. 11).

Relationship (3) — If the strength of a 6 x 12-in. concrete cylinder is taken as unity at 28 days, then the strength of the same cylinder at 1 year, $W/C = 0.7$, may be taken as 1.50 (see Fig. 9, p. 45, Reference 5). This factor is appropriate only for continuous curing, which is not usual for columns in a building. However, the British Standard Code of Practice advocates the use of the ratio 1.24 (see Reference 6, p. 26, Table 9). It seems therefore that the factor 1.30 may be employed without serious error, irrespective of the water-cement ratio.

Relationship (4) — When a 6 x 12-in. cylinder is cured by the standard method after having been cast on the same day as a concrete column, and if the strength of the concrete in the cylinder is denoted by unity, then the strength of the concrete in the column is variously given by

(a)	0.85	(see Reference 5, p. 56)
(b)	0.776	(see Reference 7, p. 101)
(c)	0.9	(see Reference 8)
(d)	> 1.0	(see Reference 9, p. 187)

It therefore seems likely that the factor 0.9 may be employed without serious error.

Relationship (4a) — The factor $0.9/1.12 = 0.8$ will be employed to relate the strength of a column to the strength of a 6-in. cube under the same conditions.

ULTIMATE LOAD ON SHORT, TIED COLUMNS WHEN SUBJECTED TO STANDARD FIRE RESISTANCE TEST

The ultimate load is given by

$$P = a\phi_c(t) \cdot F_c A_c + \phi_s(T) f_y A_s' \quad (1)$$

Where

P = ultimate load carried by a reinforced concrete column during a fire resistance test, lb

a = a constant determined by the size, age, and relation of the test sample to concrete in the actual column

$\phi_c(t)$ = a function of the time during which the column has been exposed to the fire resistance test. This function relates to a column of specific size and concrete mix. It is assumed that the function is practically independent of the age of the column, provided that it exceeds 3 months

t = time of exposure, hr

F_c = compressive strength in psi of a 6 x 12-in. concrete cylinder, cured by the standard method and equal in age to the column

A_c = net cross-sectional area of the concrete, sq in.

$\phi_s(T)$ = a function of the temperature of the reinforcement immediately prior to the collapse of the column

f_y = yield stress in psi of the reinforcement when tested at 20 C (generally f_y = 30,000 psi

A_s' = total cross-sectional area of the reinforcement

PROTECTION GIVEN TO REINFORCEMENT BY 1½ IN. OF DENSE CONCRETE

When the sole fire protection given to reinforcement is normal dense concrete, 1:2:4 mix, spalling will almost inevitably occur after about 30 min in the furnace. This is more likely to occur with columns exceeding 10 x 10 in. because with the large columns there is a greater temperature difference between the arrises and the interior. Spalling has been attributed to the pressure built up by entrapped steam which cannot readily force its way through dense concrete. The addition of light mesh reinforcement, placed about ¾ in. from the surface of the column, will reduce the amount of spalling. Despite this provision, if the cover does not exceed 1½ in. the temperature of the steel may exceed 1000 F (537 C) after approximately 2 hr of fire testing.

When this temperature is reached, the reinforcement possesses only half of its original strength and the strength will continue to fall rapidly as the temperature rises (see Reference 3, Fig. 3). It is clear therefore that with this type of concrete, and when the cover does exceed 1½ in., the contribution of the reinforcement to the total load carried is almost negligible after 2½ hr and may in fact, without the addition of the mesh, be negligible after 1½ hr (see Reference 3, pp. 22 and 76).

At 750 C (1382 F) the reinforcement possesses only 10 percent of its cold yield strength.

The ratio (R) of the load carried by the reinforcement to the load carried by the concrete is given by

$$R = \frac{\phi_s(T) f_y A_s'}{a\phi_c(t) F_c A_c}$$

The mean temperature of the concrete may be taken to be less than 400 C, and from the discussion later relating to Fig. 2 it will be seen that $\phi_c(t)$ may be assumed to be > 0.3. Hence by employing Relationship (4a), p. 1225, and taking the mean value of F_c to be 3000 psi, provided that A_s does not exceed 4 percent of the gross area of the column, we have

$$R \leq \frac{0.1 \times 30{,}000 \times 0.04}{0.8 \times 0.3 \times 3000 \times 0.96} \text{ i.e. } \leq 1/5.76$$

Hence $\phi_s(T) f_y A_s' = 0.15P$ approximately when the time exceeds 1½ hr and when no mesh reinforcement is present. This holds provided that A_s does not exceed 4 percent of the gross area of the column; a condition which generally holds in practice. Hence Eq. (1) in this case becomes approximately,

$$0.85P = a\phi_c(t) F_c A_c \quad \ldots\ldots (2)$$

DETERMINATION OF $\phi_c(t)$ FROM THE RESULTS OF FIRE RESISTANCE TESTS ON 15 X 15-IN. COLUMNS

From Eq. (1)

$$\phi_c(t) = \frac{P - \phi_s(T) f_y A_s'}{a F_c A_c}$$

And from Relationship (4a), p. 1225

$$\phi_c(t) = \frac{P - \phi_s(T) f_y A_s'}{0.8 U A_c} \quad \ldots\ldots (3)$$

U = crushing strength of a 6-in. cube of concrete cured by the standard method and equal in age to the column, psi

Note that $\phi_c(t)$ gives the ratio of the effective mean compressive strength of the concrete in the column at the time t hr to the compressive strength of the concrete immediately prior to the fire test.

Relationships (1) and (4), p. 1225, were employed to convert Eq. (1) to an equivalent equation employing cube strength instead of cylinder strength.

Table 1 shows the values of $\phi_c(t)$ redetermined from the test results given in Reference 3. It will be seen from Fig. 1 that some of the values indicated by triangles, plotted as in the reference, are questionable. Part of the discrepancy may be accounted for by the fact that the constant $0.8 = 0.9/1.12$, is used in this paper instead of 0.65 which was used in Reference 3. Another discrepancy is the fact that severe spalling in most cases probably reduced the strength of the reinforcement to a lower value than was employed in Reference 3.

The values of $\phi_c(t)$ appear to be best satisfied by the equation

$$\phi_c(t) = (1 + 0.18t)^{-3.8}$$

TABLE 1 — REDETERMINED VALUES OF $\phi_c(t)$

Page number in Reference 3	30	41	42	43	44	45	46	48	49	50	51	52
Load P carried at time of collapse, tons*	100	150	150	150	75	50	150	150	150	30	150	100
Approximate temperature T of steel prior to collapse, deg C	700	500	600	750	800	850	600	600	600	900	750	700
Fraction, $\phi_s(T)$, of cold yield strength	0.1	0.5	0.3	0.1	0.08	0.05	0.3	0.3	0.3	0.05	0.1	0.1
Load carried by steel, $\phi_s(T)f_yA_s'$ just prior to collapse, tons*	5	27	16	5	4	2.5	16	16	16	2.5	5	5
$P - \phi_s(T)f_yA_s'$	95	123	134	145	71	47.5	134	134	134	27.5	145	95
Crushing strength U of cube at time of fire test, tons per sq in.*	1.64	1.62†	2.15	2.67	1.46†	1.91‡	1.63	1.73	1.78	1.82	1.55	1.72
$0.8UA_c$	290	287	380	472	258	338	288	306	315	322	275	304
$\frac{P - \phi_s(T)f_yA_s'}{0.8UA_c}$	0.328	.429	.353	.307	.275	.141	.465	.437	.425	.085	.528	.313
Time of collapse, hr	2.08	0.78	1.30	2.68	3.30	4.13	1.18	1.23	1.18	5.97	1.08	2.05
Reference letter in Fig. 1	a	b	c	d	e	f	g	h	j	k	l	m

Note: In all the above cases, the reinforcement was supplied by four #9 mild steel bars, yield strength assumed to be 30,000 psi.

*1 ton = 2240 lb.

†These values when compared to the cube strength at 28 days, given in Reference 3, show some discrepancy. The revised values are plotted as b' and e'.

‡The cube strengths at 28 days and at 313 days are high compared to the corresponding values obtained for a similar concrete, Reference 3, p. 44.

TABLE 2—DETERMINATION OF $\phi_c(t)$ FROM RESULTS OF FIRE RESISTANCE TESTS ON 19 x 19-IN. COLUMNS

Page number in Reference 3	33	35	57	59
Load P carried at time of collapse, tons*	92	300	225	100
Approximate temperature T of steel prior to collapse, deg C	>800	670	>800	>800
Fraction, $\phi_s(T)$, of cold yield strength.	0.05	0.2	0.05	0.05
Load carried by steel, $\phi_s(T)f_yA_s'$, just prior to collapse, tons*	3	19	3	5.5
$P - \phi_s(T)f_yA_s'$	89	281	222	94.5
Compressive strength U of cube at time of fire test, ton per sq in.*	1.74	2.00	1.52†	1.72
$0.8UA_c$	495	566	433	486
$\frac{P - \phi_s(T)f_yA_s'}{0.8UA_c} = \phi_c(t)$	0.180	0.497	0.512	0.195
Time of collapse, hr	7.2	1.5	2.0	5.85
Reference letter in Fig. 2	r	s	t	u

Note: In the above results, when the cube strength at 200 days was not approximately 1.24 times the cube strength at 28 days, a correction was made.

*1 ton = 2240 lb.

†The cube strength on p. 57 when compared to the cube strength on p. 58 of Reference 3 shows a discrepancy which has been corrected.

It will be seen that the necessary condition $\phi_c(0) = 1$ is satisfied. However, infinite time of exposure is not required to give $\phi_c(t) = 0$ because after an 8-hr exposure the surface temperature of the column exceeds 2200 F. At this temperature the lowest section of the column would be unable to support the weight of the column above.

The straight line

$$\phi_c(t) = 0.5 - 0.076t \quad \text{.....(4)}$$

is a practical approximation to the plotted values from $t = 1.5$ hr to $t = 5$ hr and this is the range of values which is of particular interest.

Relationship (5) — Similarly Table 2 shows the recalculated values of $\phi_c(t)$ applicable to 19 x 19-in. columns. It is unfortunate that insufficient test results are available in this case to determine $\phi_c(t)$ more

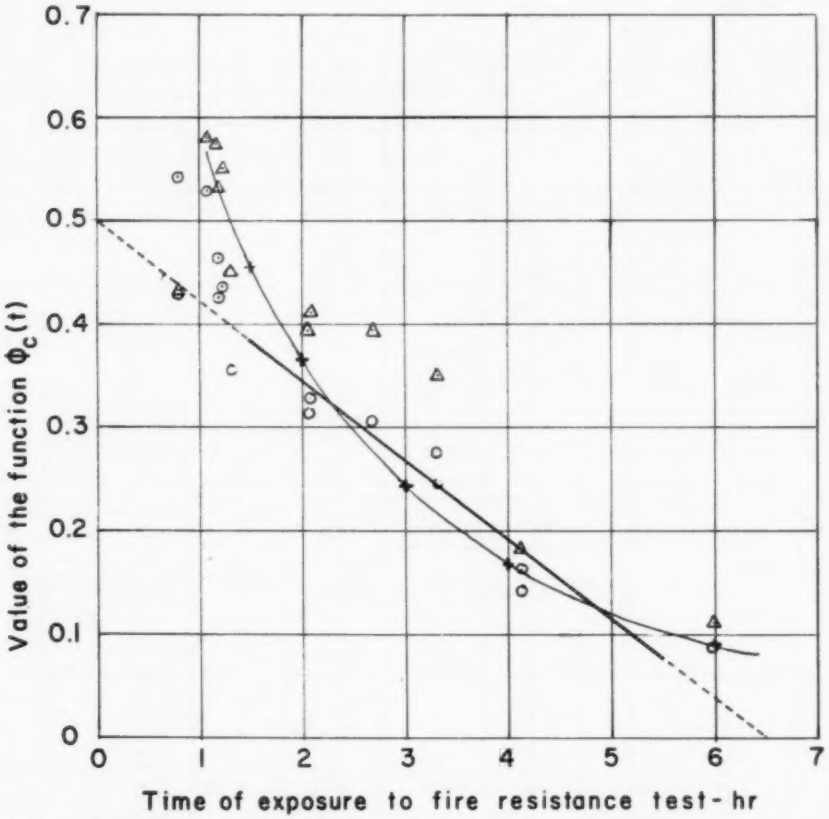


Fig. 1—Variation in the strength of concrete in a column during a fire resistance test

△ —Values of $\phi_c(t)$ given in Reference 3

O —Values of $\phi_c(t)$ for 15 x 15-in. columns given in Table 1

\+ —Values of $(1 + 0.18t)^{-4.5}$

definitely. However, from these results and those of the 15 x 15-in. columns it seems reasonable to assume that

$$\phi_c(t) = 0.62 - 0.067t$$

provided that t lies between 1.5 hr and 7 hr.

Fig. 2 shows the result of plotting $\phi_c(t)$ against t in this case.

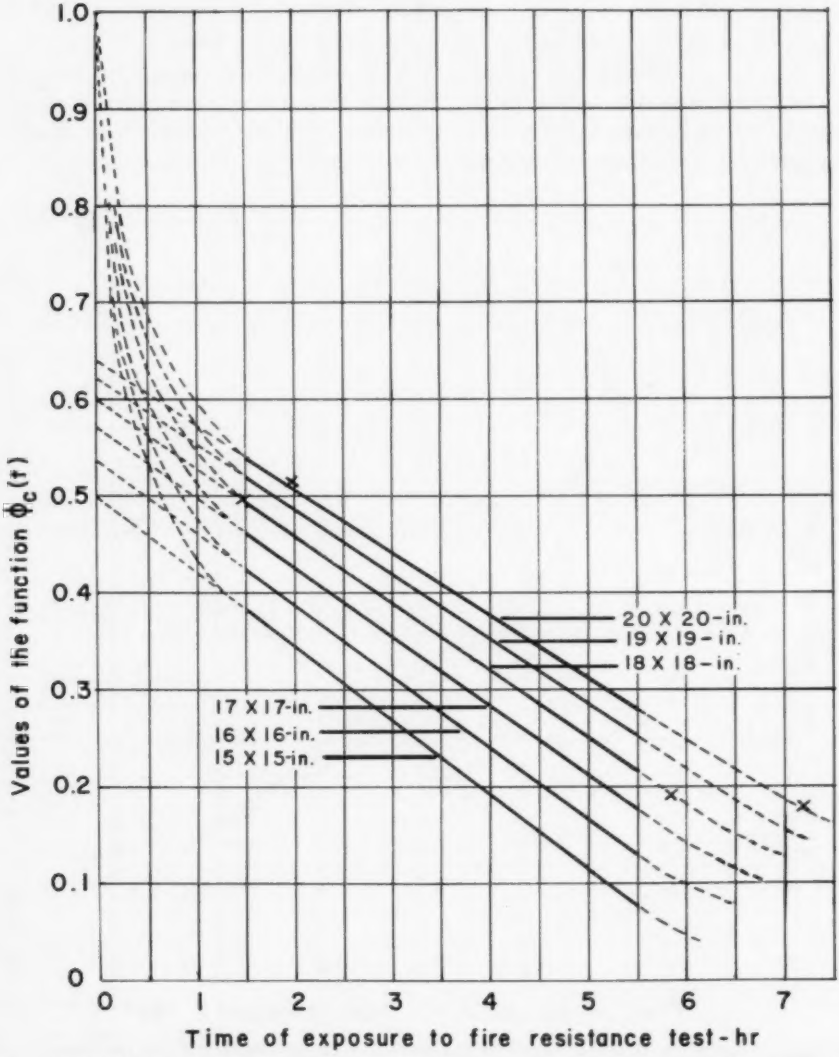


Fig. 2—Effective ratio of mean strength of concrete after a fire test to strength immediately before the test for various column sizes

X—Values of $\phi_c(t)$ for 19 x 19-in. columns from Table 2

Extension of $\phi_c(t)$ values to other column sizes

From the results obtained for 15 × 15-in. columns and 19 × 19-in. columns, interpolation gives

$$\phi_c(t) = 0.54 - 0.074t \quad \text{for 16 x 16-in. columns} \quad (5)$$
$$\phi_c(t) = 0.57 - 0.072t \quad \text{for 17 x 17-in. columns} \quad (6)$$
$$\phi_c(t) = 0.60 - 0.070t \quad \text{for 18 x 18-in. columns} \quad (7)$$

and extrapolation gives

$$\phi_c(t) = 0.64 - 0.065t \quad \text{for 20 x 20-in. columns} \quad (8)$$

All of these relationships are shown in Fig. 2. Note that as the size of the column increases, the straight lines crowd closer together. This is due to the fact that an extra inch in the size of a column has less effect on the effective mean strength $\phi_c(t)$ than if the inch were added to a smaller column.

ESTIMATION OF FIRE RESISTANCE OF A COLUMN, COVER NOT EXCEEDING 1½ IN.

The following method is introduced for estimating the fire endurance of columns made from dense concrete.

Put $\phi_c(t) = h - kt$, where h and k are determined by Eq. (4) to (8). Then provided that 1½ hr $< t <$ 6 hr from Eq. (3)

$$t = \frac{h}{k} - \frac{2240P - \phi_s(T) f_y A_s'}{0.8kUA_c} \quad (9)$$

Since for $t >$ 1½ hr, $\phi_s(T) f_y A_s'$ is negligible

$$t = \frac{h}{k} - \frac{2240P}{0.8kUA_c} \quad (10)$$

Eq. (10) is only approximate and in general gives conservative values of t. If the percentage of reinforcement exceeds 2.5 percent, it is advisable to employ Eq. (9).

When the percentage of the reinforcement exceeds 2.5 percent but is less than 4 percent, then for this depth of cover (1½ in.), P in Eq. (10) may be replaced by 0.85 P.

By substituting the values of h and k from Eq. (4) to (8) into Eq. (10) we have

$$t = 6.58 - \frac{36{,}800}{UA_c} P \text{ hr} \quad \text{for 15 x 15-in. columns} \quad (11)$$

$$t = 7.31 - \frac{37{,}800}{UA_c} P \text{ hr} \quad \text{for 16 x 16-in. columns} \quad (12)$$

$$t = 7.92 - \frac{38{,}900}{UA_c} P \text{ hr} \quad \text{for 17 x 17-in. columns} \quad (13)$$

$$t = 8.57 - \frac{40{,}100}{UA_c} P \text{ hr} \quad \text{for 18 x 18-in. columns} \quad (14)$$

$$t = 9.26 - \frac{41{,}500}{UA_c} P \text{ hr} \quad \text{for 19 x 19-in. columns} \quad (15)$$

$$t = 9.85 - \frac{43{,}100}{UA_s} P \text{ hr} \quad \text{for 20 x 20-in. columns} \quad (16)$$

These values may be related to American and Australian practice by replacing U with 1.12 F_c, where F_c is the compressive strength of a 6 x 12-in. cylinder, cured by the standard method and equal in age to the column just prior to the fire resistance test. A further simplification may be made, if the cross-sectional area of the reinforcement ≤ 4 percent of the gross area of the column, by assuming the mean percentage of $A_s = 2$ percent. Hence if P is the applied load in tons (2240 lb)

$$t = 6.58 - 148\ P/F_c \text{ hr} \quad \text{for 15 x 15-in. columns} \quad (17)$$

$$t = 7.31 - 134\ P/F_c \text{ hr} \quad \text{for 16 x 16-in. columns} \quad (18)$$

$$t = 7.92 - 122\ P/F_c \text{ hr} \quad \text{for 17 x 17-in. columns} \quad (19)$$

$$t = 8.57 - 112\ P/F_c \text{ hr} \quad \text{for 18 x 18-in. columns} \quad (20)$$

$$t = 9.26 - 105\ P/F_c \text{ hr} \quad \text{for 19 x 19-in. columns} \quad (21)$$

$$t = 9.85 - 98\ P/F_c \text{ hr} \quad \text{for 20 x 20-in. columns} \quad (22)$$

It is believed, based on the information available at the time of writing this paper, that the above formulas are at present the most suitable. The compressive strength of the concrete at 28 days (f_c') may of course be substituted for F_c, which in the above equations refer to the strength at an age of about 8 months or more [see Relationship (3), p. 1225].

Designers will particularly note that a marked increase in the fire endurance is theoretically obtained by combining a relatively light load with a concrete of high strength. Attention to this detail may appreciably reduce the size of columns required for a projected building. Further test data will be necessary before this story can be fully substantiated.

Introduction of a fire exposure safety factor into the assessment of fire endurance

If a column collapses after a 4-hr exposure to the fire resistance test with an applied load of 100 tons, it is naturally assumed that the actual load in practice will be less than 100 tons. The 100-ton load may be a necessary assumption, however, when it is considered that during a fire many hundreds of tons of water may be poured on to floors by firemen. In addition, following a fire, further deterioration of the concrete occurs. Tests indicate that if a load of P lb is borne by a column until it collapses at 4 hr, then with a load of about 80 percent P and with the fire test terminating at 4 hr, there is a strong probability that this column would fail to support the load for more than 2 days while the column is cooling.

Thus, in deciding on the value P to employ in Eq. (17)-(22), a suitable safety factor must be incorporated, as the test results from Reference 3 have not taken the above effect into consideration. This aspect is primarily the work of legislating authorities.

CASE WHEN ADDITIONAL PROTECTION IS GIVEN TO REINFORCEMENT

When columns with dimensions less than 18 x 18 in. have the cover to the reinforcement increased to 2½ in. of dense concrete, the reinforcement will be unlikely to possess more than 50 percent of its cold yield strength during the whole period of a 3-hr fire test or 30 percent of its cold yield strength at 4 hr, even if mesh reinforcement is added. Columns with a cross-section area not less than 18 x 18 in. present a larger surface area to the hot furnace gases and hence, with the same cover, the temperatures of the reinforcement will be lower at equivalent times as previously. Hence for these larger columns, the reinforcement will possess approximately 50 percent of its cold yield strength at 4 hr, but it would be advisable to employ light mesh reinforcement as a precaution unless the column is 20 x 20 in.

In the case where the side of the column is less than 18 in., the thickness of the cover must be increased to 3 in., preferably with the addition of mesh reinforcement, if it is desired that the reinforcement should possess at least 50 percent of its cold yield strength throughout a 4-hr fire resistance test.

The cover of 2½ in. of dense concrete required for the larger columns is equivalent to a cover of less than 2½ in. of concrete possessing a lower thermal diffusivity. It is unfortunate that some aggregates which impart a relatively low thermal diffusivity to concrete, e.g., vermiculite, also produce low strength concrete. However, a useful reduction in the thermal diffusivity can be obtained by employing as aggregates expanded clay or shale, blast furnace slag, and clean crushed bricks, without incurring a serious loss in strength. Designers should seek expert advice when contemplating the use of these aggregates to obtain 3000 psi at 28 days. Lack of knowledge may lead to the production of inferior concrete, which may even deteriorate in the course of time. Apart from the added protection to the steel, these concretes provide increased insulation to the core of a column. Hence the effective strength of a column, during a fire resistance test, is increased. The aggregates mentioned as alternatives in this paragraph may be used to give a cover of 2⅛ in. to replace 2½ in. of dense concrete.

Where the column size must be reduced still further, a cover of 1½ in. of these alternative concretes may be used to give adequate protection to the reinforcement for 4 hr in a fire resistance test, provided that the correct minimum thicknesses of surface treatments or render plasters are also applied. One of the best treatments is given by adding 1½ in. of neat gypsum plaster which, of course, has the advantage of a hard finish. Other treatments equally effective are: ½ in. of vermiculite or perlite plaster; ⅝ in. of certain proprietary acoustical plaster; ⅝ in. of sprayed asbestos; and ¾ in. of 1:2 sanded gypsum plaster.

The 3-in. cover of dense concrete, required for columns less than 18 x 18 in., may be replaced by a 2½-in. cover of one of these alternative concretes. If the column size must be kept to a minimum and still give adequate protection to the reinforcement for 4 hr during a fire resistance test, a 1½-in. cover of one of these alternative concretes may be supplemented by the addition of ½ in. of neat gypsum plaster. The thicknesses of equivalent treatments are given in the preceding paragraph.

Some doubt might be raised regarding the apparently conservative thicknesses of some of the alternative plasters or surface treatments. The author believes that a conservative view must be taken of certain test results in view of the remarks in the second paragraph of the introduction. Furthermore, it is not certain that some treatments will continue to adhere to a column or retain their effectiveness after several years in service.

On the assumption that sufficient protection has been given to the reinforcement to enable it to possess half the cold yield strength immediately after the expiration of a 4-hr fire test, it will be seen that the mean temperature within a column will be appreciably lowered if a surface treatment has been applied to the column. The value of A_c is of course unaltered by such an application but the value of $\phi_c(t)$ will be increased. If the reinforcement is placed deeper within the column there will still be a significant increase in the value of $\phi_c(t)$ without the application of a surfaced treatment. When the procedure for obtaining Eq. (2) is followed, it will be seen from Fig. 2 that $\phi_c(t) \geqq 0.2$, but adopting the modification, which has just been discussed, it is reasonable to assume $\phi_c(t) > 0.3$ at this time; hence

$$R \leq \frac{0.5 \times 30{,}000 \times 0.04}{0.9 \times 0.3 \times 2500 \times 0.96}$$

Therefore

$$R \leq 0.93$$

$$P - \phi_s(T) f_y A_s' \geqq 0.5\,P$$

And obviously

$$P - \phi_s(T) f_y A_s' \leq P$$

Hence it may be assumed for practical purposes without serious error that

$$P - \phi_s(T) f_y A_s' = 0.75\,P \text{ (approximately)}$$

Or

$$t = \frac{h}{k} - \frac{2240 \times 0.75\,P}{0.9\,F_c A_c k \times 0.98}$$

Thus Eq. (17) to (22) become:

$$4 = 6.58 - 111\ P/F_c \quad \text{for 15 x 15-in. columns} \quad (23)$$

$$4 = 7.31 - 100\ P/F_c \quad \text{for 16 x 16-in. columns} \quad (24)$$

$$4 = 7.92 - 92\ P/F_c \quad \text{for 17 x 17-in. columns} \quad (25)$$

$$4 = 8.57 - 84\ P/F_c \quad \text{for 18 x 18-in. columns} \quad (26)$$

$$4 = 9.26 - 79\ P/F_c \quad \text{for 19 x 19-in. columns} \quad (27)$$

$$4 = 9.85 - 74\ P/F_c \quad \text{for 20 x 20-in. columns} \quad (28)$$

If a load P is assigned for a column, the minimum requisite cylinder strength needed for a 4 hr fire resistance can be determined. Note that F_c relates to the strength of a concrete cylinder whose concrete is equal in age to that of the corresponding column.

As an illustration of these formulas, if the reinforcement in a 20 x 20-in. column is to have a 2½-in. cover of dense concrete and the load, including the safety factor, is to be 200 English tons, then $F_c = 2530$ psi, approximately. This value can be modified, according to the standard practice of the country concerned, so as to give the requisite 28-day cylinder strength.

MAXIMUM TEMPERATURES ATTAINED WITHIN REINFORCED CONCRETE COLUMNS GREATER THAN 14 X 14 IN.

As was discussed earlier, calculated values of the temperatures within a reinforced concrete column during a fire resistance test are unreliable. Therefore, recourse has been made to experimental values of the maximum temperatures occurring within the columns at various points during a test. Fig. 3 shows the maximum temperatures attained during fire resistance tests on 15 x 15-in. and 19 x 19-in. columns made of dense concrete having siliceous aggregate.

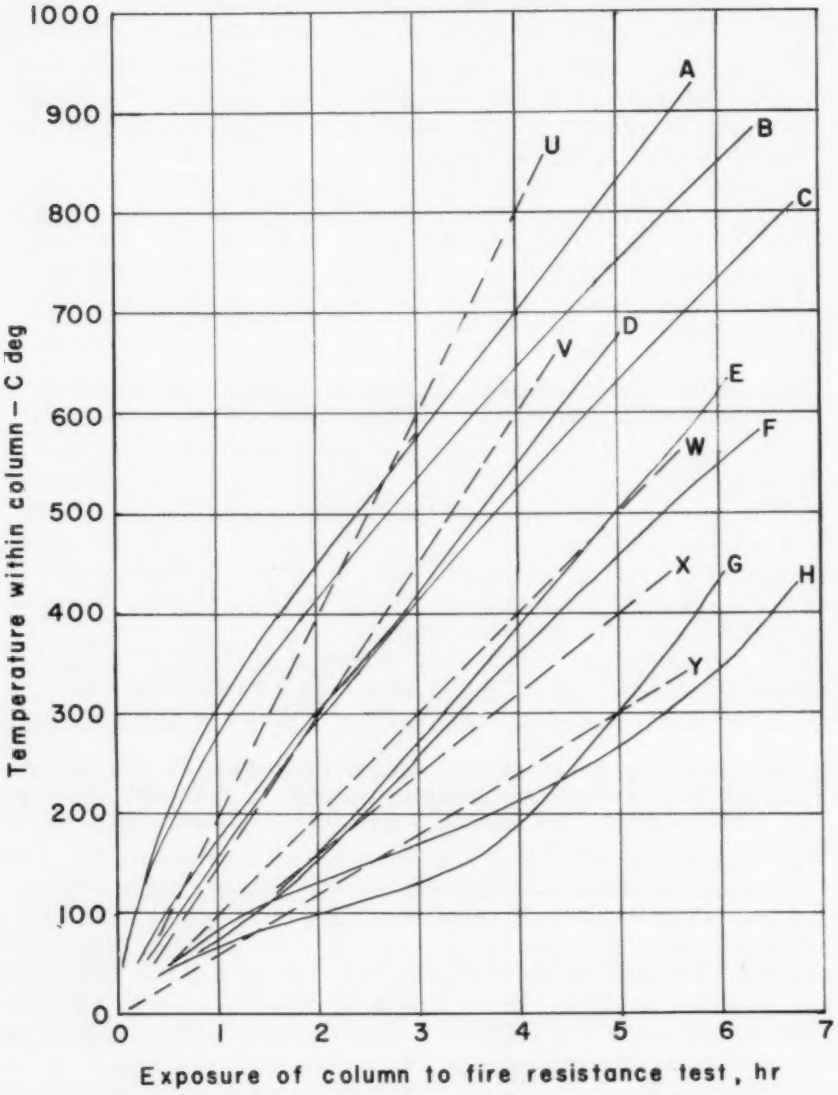
From approximately 2 hr to 4 hr during these tests it is possible to represent the time-temperature relationship satisfactorily by a straight line. Lines U, V, W, X, and Y in Fig. 3 were drawn in this manner.

DETERMINATION OF EFFECTIVE MEAN COMPRESSIVE STRENGTH OF CONCRETE WITHIN A COLUMN DURING A FIRE RESISTANCE TEST FROM TEST RESULTS ON SMALL CYLINDERS

The most reliable estimate of the effective mean compressive strength of concrete subjected to a fire resistance test is unquestionably obtained from tests which simulate the actual conditions in a building fire. These estimations are more accurate than any of those obtained from tests conducted in a small furnace on cylinders 2 in. in diameter and 4 in. long. However, an alternative approach is desirable, mainly for economic reasons, when it is required to investigate the relative merits of various types of concrete subjected to various curing periods.

The Building Research Station in England made a series of tests on 2 x 4-in. cylinders (see Reference 10, pp. 85-94). These cylinders were tested under different conditions and three relationships were obtained between the mean temperature T C of the cylinder and the ratio of the compressive strengths at T C and 20 C.

(a) The cylinders were tested hot and under a working stress of 1040 psi, i.e., when an assigned temperature was reached the load was increased until failure occurred.



(b) An alternative to (a), which was not carried out, would be to allow the cylinders to cool for 24 hr, after an assigned temperature had been attained, and then to increase the load until failure occurred.

(c) The cylinders were not loaded during the heating period, but were loaded to failure immediately upon attaining an assigned temperature.

Legend

Line designation letter	Distance from center of column face, in.	Column size, in.	Reference 3
	Based on fire tests		
A	1¾	15 x 15	p. 50
B	1	19 x 19	p. 33
C	2½	19 x 19	p. 33
D	2½	15 x 15	p. 50
E	3½	15 x 15	p. 50
F	3½	19 x 19	p. 33
G	5½	15 x 15	p. 33
H	5½	19 x 19	p. 50
	Based on calculations		
U	1½	14 x 14	—
V	2½	14 x 14	—
W	3½	14 x 14	—
X	4½	14 x 14	—
Y	5½	14 x 14	—

Fig. 3—Maximum temperature within reinforced concrete column at various depths, dense concrete 1:2:4 mix

(d) The cylinders were tested as in (c) but were allowed to cool for 24 hr, after the desired temperature had been attained, before loading to failure.

Fig. 4 shows the results of the above tests and includes the estimated position of the curve corresponding to Case (b). It has been assumed that Case (b) is related to Case (a) in approximately the same manner as Case (d) is related to Case (c).

It is possible that the termal stresses which were incurred during these tests were significantly and relatively less than would be incurred with cylinders approaching the size of actual columns. This discrepancy is due to the fact that it is possible to obtain uniform temperatures throughout small cylinders treated under laboratory conditions, but the reverse occurs in columns by the very nature of a fire resistance test. Thus, each curve in Fig. 4 should perhaps be reduced in ordinate to approximate more closely the full-scale conditions. The multiplier 0.7 is tentatively suggested.

Relationship (6) — The author believes that Case (b) is of the greatest practical importance if the collapse of a building is to be avoided after the fire has been extinguished. Thus the following theory deals with this case, and if F_c is the crushing strength of a cylinder before heating, then $(1 + aT - bT^2)F_c$ will be assumed to be the crushing strength of a similar cylinder treated as in (b) above. The constants a and b can be determined from test results.

The difficulty in performing the appropriate integration when dealing with the columns has been avoided by associating with each ring of 1-in. width, in the cross-sectional area of the column, a mean strength as-

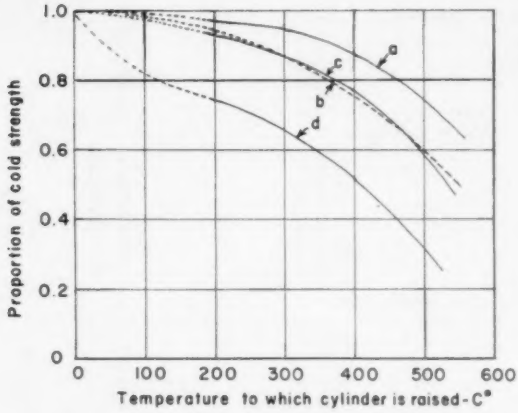


Fig. 4—Reduction in strength of 2 x 4-in. concrete cylinders heated under various conditions

sociated with the temperature on the circle in the center of the ring (see Fig. 5). The isotherms may be taken as concentric circles without undue loss in accuracy, even though the isotherms become closer to squares as the center is approached. This leads to a slightly lower estimate being made of the compressive strength.

If r = radius of inscribed circle within the column in inches, the formula $4A_0 = 0.86\ r^2$ sq in. is the area of the four corner fillets while the following formulas give the area for each isothermic ring.

$$A_1 = \pi\,(2r - 1)$$
$$A_2 = \pi\,(2r - 3)$$
$$\vdots \qquad \vdots$$
$$A_6 = \pi\,(2r - 11)$$
$$A_7 = \pi\,(r - 6)^2$$

The contribution to the strength of the concrete within the column given by the four corner fillets, Area $4A_0$, may be neglected as the reinforcement takes up part of this area and the concrete within this area, which received heat flow from two sides, is at such a high temperature that its strength is very low. Furthermore, spalling at the arrises is likely to occur which leads to still higher temperatures.

The ultimate load C in pounds carried by the concrete from Relationship (4), p. 1225, is given by

$$C = 0.9F_c\left[(1 + aT_1 - bT_1^2)A_1 + (1 + aT_2 - bT_2^2)A_2 \cdot \cdots + (1 + aT_7 - bT_7^2)A_7\right]$$

where T_1 is the mean temperature in centigrade within the ring area A_1 and a and b are constants determined from Fig. 4, with similar meanings for T_2 through T_7.

$$C = 0.9F_c\Big[(1 + aT_1 - bT_1^2)\,\pi\,(2r - 1) + (1 + aT_2 - bT_2^2)\,\pi\,(2r - 3)$$
$$\cdots + (1 + aT_7 - bT_7^2)\,\pi\,(r - 6)^2\Big]$$

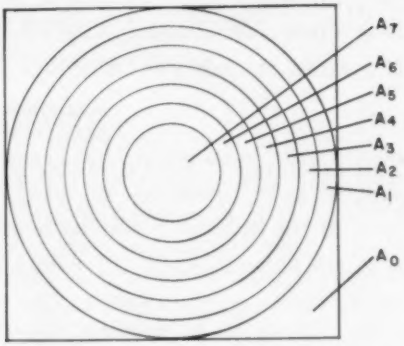


Fig. 5—The construction of approximate isotherms within a column of reinforced concrete

The mean temperature, taken at a depth of ½ in. from the surface in the A_1 ring, will be increased by the conditions existing in the A_0 areas and by the formation of fine surface cracks resulting from the steep thermal gradient near the surface. Thus after an interval of 2 hr, when the mean temperature in A_1 will exceed 600 C, the relatively small contribution to the strength of the column given by A_1 may be neglected. Apart from the foregoing thermal considerations, the contribution to the load-bearing capacity of the column offered by the outer shell is neglected by some designers since the load may unavoidably be applied nonuniformly over the area at the top of the column. The strength of the concrete within the circle area A_7 may be taken as F_c as the mean temperature within this area is unlikely to exceed 200 C before at least 5 hr, provided that the side of the column exceeds 15 in.

$$C = 0.9\pi F_c \left\{ \begin{array}{r} (r-1)^2 + a\left[(2r-3)T_2 + (2r-5)T_3 + \cdots\cdots (2r-11)T_6\right] \\ -\, b\left[(2r-3)T_2^2 + (2r-5)T_3^2 + \cdots\cdots (2r-11)T_6^2\right] \end{array} \right\}$$

From Fig. 3

$$\begin{aligned} T_2 &\leqq 200t \\ T_3 &\leqq 150t \\ T_4 &\leqq 100t \\ T_5 &\leqq \ 80t \\ T_6 &\leqq \ 60t \end{aligned}$$

where t is in hours. Hence

$$C \geqq 0.9\pi F_c \left\{ (r-1)^2 + a\,t\,(1180r - 3430) - 1000\,b\,t^2\,(165r - 400) \right\} \quad \ldots\ldots (29)$$

From Fig. 4, the constants a and b for Curve (b) are given by

$$\begin{aligned} a &= 1.15 \times 10^{-4} \\ b &= 1.85 \times 10^{-6} \end{aligned}$$

Further work may result in revised values for these constants, especially when the effect of the relation of the thermal stresses in the small cylinders to the stresses in a full-scale test are considered.

Substituting for a and b will enable the value of C to be found for any assigned value of t. We may, however, proceed to obtain a formula for t by rewriting Eq. (29) as a quadratic equation in t. Then

$$t^2 - \frac{a(1180r - 3430)}{1000b(165r - 400)}\,t + \frac{C}{900\,\pi\,F_c b(165r - 400)} - \frac{(r-1)^2}{1000b(165r - 400)} = 0$$

If a mean column size, side 17½ in., is taken, $r = 8.75$ in.

$$\frac{a(1180r - 3430)}{1000b(165r - 400)} = 0.41$$

And assuming $C = 200{,}000$ lb, $F_c = 3500$ psi, the constant term will equal −38.65. Thus for the range of constants considered, the second term of the quadratic may be neglected. The solution of $x^2 - px - q = 0$ is given by $x = \frac{1}{2}p \pm \sqrt{q}$ where $p << q$

$$t = \sqrt{\frac{2.83(r-1)^2F_c - C}{0.00523F_c(165r - 400)}} + 0.2 \text{ approximately} \quad \text{.....(25)}$$

This formula will now be applied to some of the test results given in Reference 3.

(a)
$F_c = 5320$ psi, at 434 days (from p. 43)
$C = 145$ tons $= 325{,}000$ lb (from Table 1)
$r = 7.5$ in.
$\therefore t = 3.86$ hr

The actual test result gave 2.7 hr

The theoretical result must, in this case, be multiplied by the factor 0.7 to obtain the test result. The discrepancy may be the result of compressive strength deduced from small cylinders being too high due to the low thermal stresses developed in these cylinders.

(b)
$F_c = 3280$ psi, 292 days (from p. 30)
$C = 95$ tons $= 213{,}000$ lb (from Table 1)
$r = 7.5$ in.
$\therefore t = 3.72$

The actual test result gave 2.1 hr. The multiplying factor is reduced to 0.565.

(c)
$F_c = 3440$ psi, 251 days (from p. 59)
$C = 94.5$ tons $= 212{,}000$ lb (from Table 2)
$r = 9.5$ in.
$\therefore t = 5.0$ hr

The actual test result gave 5.85 hr.

In this case it must be remembered that when the exposure to the fire resistance test exceeds 4 hr, the straight line approximations to the temperatures give far higher temperatures than is actually the case. This effect probably more than compensates for the lack of comparable thermal stresses discussed earlier.

It will be seen, therefore, that this second method must be treated with reservations, but it is probable that when more exact values are assigned to the constants a and b in Relationship (6), and considering

also the relationship of Curve (b) in Fig. 4 to the other curves, the method will be of greater value than it is at present. Further investigations are at present being carried out in Australia.

CONCLUSION

Proposals have been advanced in this paper for reviewing the present method of dealing with the fire resistance requirements for reinforced concrete columns in a proposed building. If the results of further fire tests confirm this theory, the methods advocated should enable a designer to comply with the requirements of various building codes and yet to economize in the use of steel and concrete. If opinions are expressed that the methods given are not conservative, it should be remembered that the safety factor may be chosen to satisfy any legislative authority. Moreover, the argument presented in the introduction may induce some authorities to relax fire resistance requirements where it is considered that these requirements are too conservative. This is more especially the case with reinforced concrete columns. The methods presented in the paper may be extended readily to deal with circular columns and other types of reinforcement.

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Title No. 56-62

Multipurpose Building of Precast Thin-Shell Panels

By ARSHAM AMIRIKIAN

An assembly of simple precast panels, connected together by bolts and welded inserts, provides the structural framing of a single-story gable-roof building, suitable for many uses. The panels are ribbed, consisting of a thin slab and peripheral stiffening members. Arranged for 4-ft modular framing, two pairs of panels—one pair placed in the roof and the other in the sidewalls—compose a building segment 4 ft wide.

Framing arrangement, design, assembly, and jointing are discussed. Information is also given on strength tests and service behavior of an experimental building built of these panels.

According to a recent marketing survey, there is an increased demand for simple, one-story prefabricated buildings. They are used extensively by the industry and in farming as warehouses, shops, storage sheds, and for general utility purposes. Structures of this type, presently available on the commercial market, are generally of light-gage steel.

There have been some attempts to produce similar buildings of prefabricated concrete framing. For various reasons, however, a successful prototype, possessing truly competitive merit, has not as yet reached the market. In analyzing this condition, some have found the needed promotional effort lacking. Others have traced the cause to such unfavorable factors as the absence of an efficient design, bulkiness, and many difficulties associated with the casting, handling, transportation, assembly, and jointing of the concrete framing.

Long experience with the use of precast concrete in the construction of the U.S. Navy's shore structures has shown that at least one framing element can be mass produced at an attractively low cost. That is the so-called thin-shell ribbed panel. Such panels have been widely utilized as local framing members in roofs, floors, and walls, often in conjunction with conventional types of main framings. This suggests that if a design can be developed in which the entire framing would consist of panels of this type, the resulting building would possess great economic merit.

While productionwise the idea of an all-panel building appears quite appealing, the success of application involves a number of problems in design, assembly, and jointing which require special study and some experimentation. These problems are discussed here in detail, especially in connection with an actual design.

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GENERAL ARRANGEMENT

Building shape

Prefabricated metal buildings currently in production conform to several cross-sectional shapes. Most have vertical walls, with flat, gabled, or curved roofs, and some are of semicylindrical form. While concrete is equally adaptable for prefabricating framings of these and other shapes,* fabrication experience has indicated that the production of framing components conforming to curved or angular outlines, which may be referred to as "shaped" outlines, would be quite costly. Hence, for casting economy, the main element of framing must have a reasonably simple form. On the other hand, in structural efficiency, a shaped outline is more advantageous than a rectangular or "simple" outline. A third consideration concerns the functional or usability aspects of the building outline. In this consideration, the preferable outline is one which results in a maximum usable floor area for a given amount of closure framing.

Obviously, none of the shapes would provide an equally impressive advantage on all three bases of comparison. For what is considered superior structurally, or functionally, may be found more costly for production. Conversely, a simply shaped framing that is more economical to fabricate may develop comparatively lower strength or resistance than that of a shaped contour. The choice of the framing will then have to be made on the basis of a parallel study where appropriate factors are applied to the various items in the over-all cost estimate.

A study of this problem, made by the writer in connection with a Navy requirement for a prefabricated concrete building to serve a number of purposes, resulted in choosing an arrangement consisting of vertical walls and a low-gable roof.

In structural efficiency, a gable frame may be considered as a compromise between an arch and a rectangular bent. The height of the gable, or the roof slope, may be so regulated that under the normally predominant load condition a favorable stress path or distribution is obtained. Functionally, the arrangement results in an enclosure over a fully usable floor area and good clearances.

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*Amirikian, Arsham, "Precast Concrete Offers Protection Against Atomic Blast," ACI JOURNAL, V. 22, No. 7, Mar. 1951 (*Proceedings* V. 47), pp. 497-516.

Framing components

The choice of a proper outline is only a part of the framing problem. After that is determined, a decision must be made regarding the subdivision of the assembly into its framing components. In conventional precast construction, there is a series of main frames and a system of secondary frames, which also consists of local elements and supporting members. In our particular application of the precasting technique, all three of these components are merged into a single unit. An assembly of such units then constitutes the total building.

Framing module

For a rectangular layout, such as shown in Fig. 1, the framing units can be treated in two groups: (a) units which form the side walls and roof, and (b) those used in the end walls. The width of the units in

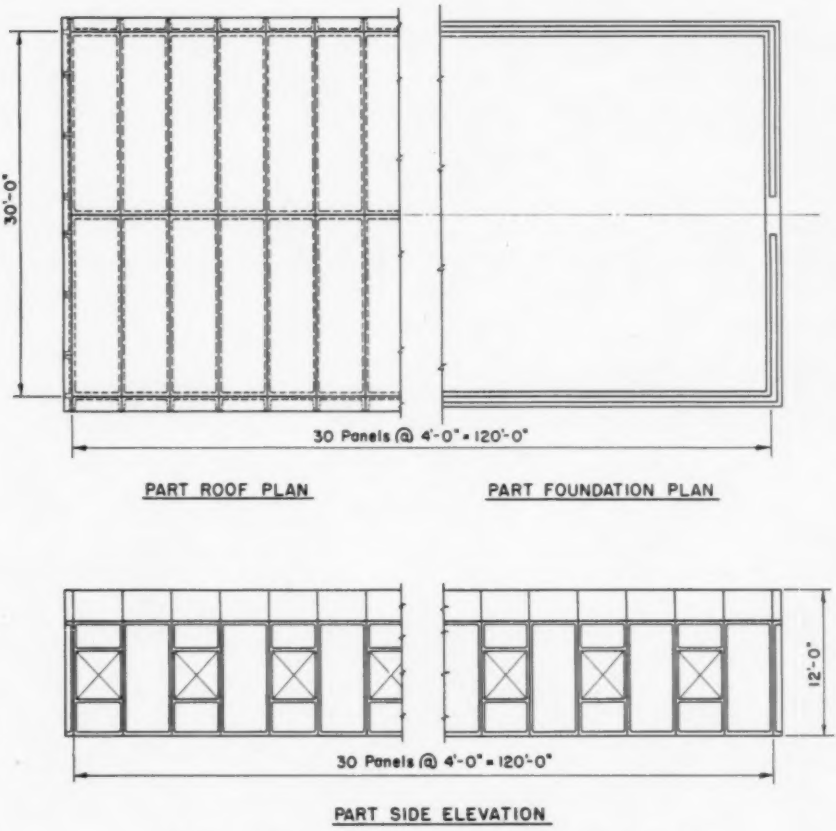

Fig. 1 — General layout of modular panel building

the first group represents the basic dimension, or module, of the framing. This may vary from 2 to 8 ft or more, depending on factors of arrangement and productiton. While a small module will provide greater flexibility in fitting the unit into a given layout, and will result in relatively smaller and lighter castings, its use will necessitate handling a greater number of panels for the same enclosure and a correspondingly large amount of jointing and sealing. Based on experience gained to date, a 4-ft module appears to be a good workable dimension. The same module is also applicable to the panels in the end walls, although at these locations some deviation may be necessary because of the specified width of the building or the end doors.

Splices in framing assembly

In the all-panel concept of framing, four panels are put together to shape the transverse cross section of the building, as shown in Fig. 2. Of these, two panels serve as vertical side walls, and the other two, placed at the desired gable slope and surmounting the walls, form the roof. The arrangement thus requires a longitudinal splice at the roof apex, and one along each eave. In addition, there will, of course, be transverse joints between adjacent panels at modular intervals.

DESIGN AND FRAMING DETAILS

The various details of framing can best be described by an illustrative example. For this purpose use will be made of the actual design of an experimental building, built for strength tests and service behavior observations. It is a 30-ft span utility building, shown in plan and side elevation in Fig. 1, and in cross-sectional view in Fig. 3.

Basis of design

For design purposes, the four-panel assembly in Fig. 2 may be considered as a rigid frame. There are five joints in the assembly. One at

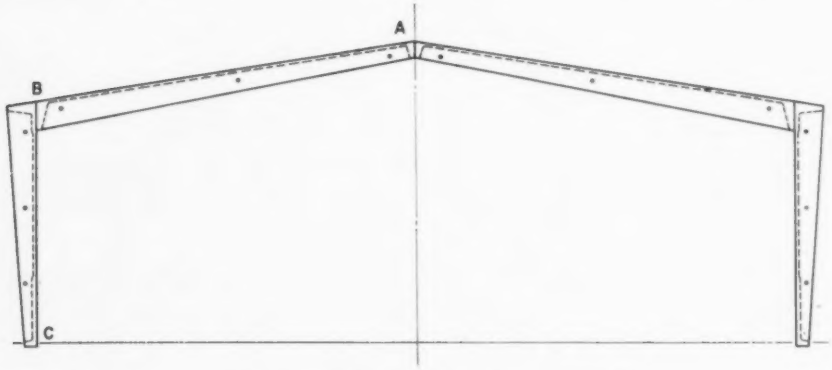

Fig. 2 — Four-panel building assembly

roof center (A), and one at each eave, (B), and wall base (C). However, for stability and frame action, only two of the five joints, namely, those at the two eaves, need be rigid or moment resisting. Under this assumption, the assembly then becomes a statically determinate three-hinged bent, requiring no special analytical treatment. In reality, since no mechanical hinges are used, there will be some moments developed

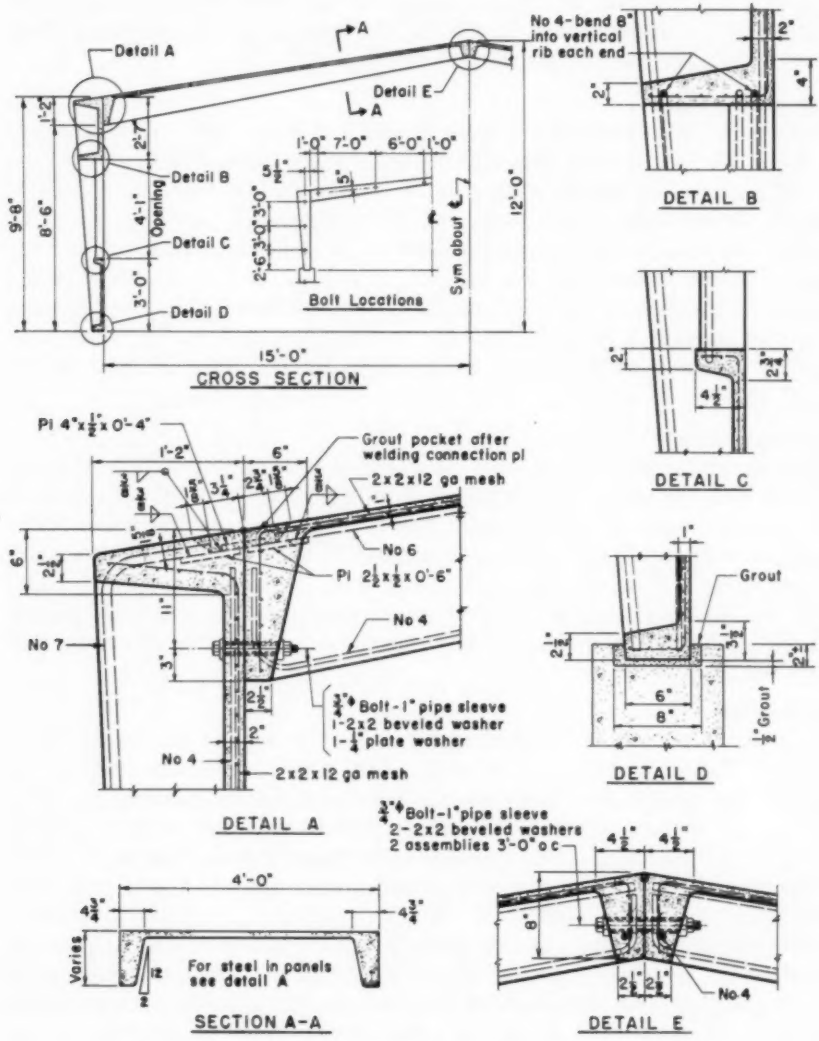

Fig. 3 — Roof and side wall panel framing and details

at these so-called hinged or moment-free joints; but this is a conservative and simplifying assumption.

Roof panels

The roof panels, measuring 4 ft by about 15 ft, consist of a 1-in. thick skin slab and four edge ribs. The rib at the upper end is 8 in. deep, that at the lower end 14 in., and those on the two sides have tapering depths, varying from 8 in. to 14 in. The side ribs constitute the primary-strength members of the panel, while the end ribs serve as edge stiffeners for the slab and as struts in the assembly. In addition, they provide flanges for the interbolted connections.

Side wall panels

The side wall panels are 4 ft wide and 9 ft 8 in. high. They also are of the ribbed type, with side ribs tapering from 14 in. at the top to 6 in. at the bottom. In panels with window openings, the slab is 2 in. thick; and in those without windows, the 2 in. thickness prevails from the bottom to window sill height, as well as in the flange strip of the top edge rib; the balance of the slab is only 1 in. thick. In the framing assembly, the ribs of the wall panels project outward; thus the tapering side ribs present a pleasing exterior appearance, and at the same time a clear interior wall surface is provided by the smooth face of the panel slab.

End wall panels

The arrangement of end wall panels is shown in Fig. 4. For the end with a door opening, two schemes are indicated. A centrally located 3-ft door is used in one scheme, and a 13-ft door in the other. The balance of the 30-ft end opening is enclosed with a set of typical interior panels each 5 ft wide, two end panels 4 ft 8 in., and a special panel over the door opening. In detail they are similar to the side wall panels, except that the top edge ribs are placed on a slope to coincide with the roof line.

Connections

The basic device for interconnecting the panels in the assembly is bolting. For this purpose a series of 1-in. pipe sleeves are cast in the ribs of the panels for ¾-in. bolts. For longitudinal continuity, three sleeves are cast in each side rib of roof and wall panels, with a spacing as indicated in Fig. 3. For transverse continuity, two sleeves are provided in each end rib of the roof panels and in the thickened web near the top of the wall panels.

The bolted connection at the eaves is reinforced with a welded strap, the detail of which is shown in Detail A of Fig. 3. This is the main joint of the framing, and the required flexural strength can best be provided through welding. The detail is relatively simple. It consists of an anchored steel insert, recessed in the top corner, and an overlapping

splice plate which is welded in place after assembly and alignment of the panels.

Bolted connections are used also for joining the end wall panels to each other and to the roof panels.

Only grout is used to connect the wall panels to the foundations.

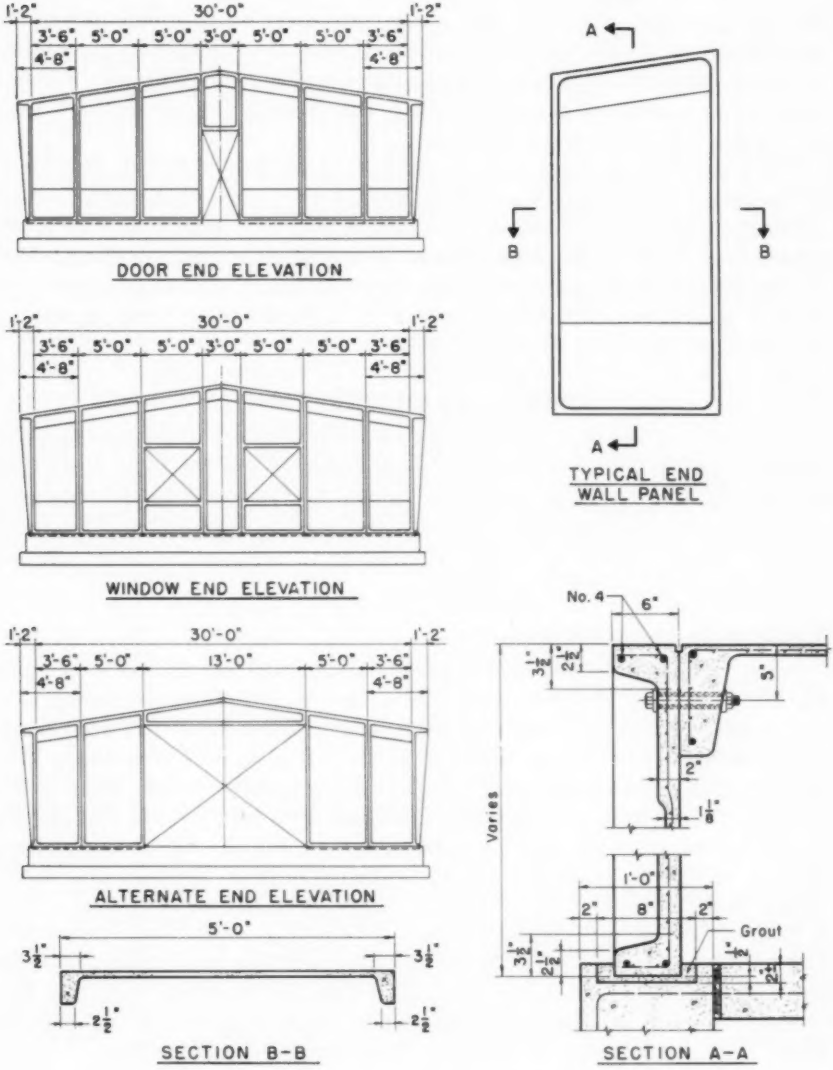

Fig. 4 — Arrangement of panels in end walls

As will be noted from the details in Fig. 3 and 4, a 2-in. deep recess is provided in the foundation pad for this purpose. The arrangement will permit the shimming of the base for levelment of the panels prior to grouting of the joint.

Sealing of joints

The use of roofing or siding is not contemplated in a panel building. For all practical purposes, the concrete panels in themselves may be considered as watertight. The problem here then concerns the sealing of joints between the panels. This is accomplished by calking. For this purpose a groove is provided along the joining edges of the panels to contain the calking compound.

A typical joint seal detail is shown in Section B-B of Fig. 5. It consists of a rectangular recess ¾-in. deep and about ¾-in. wide, filled with a calking compound. At the welded connection, also shown in Fig. 5, the recess is of trapezoidal cross section. Here, to economize in the amount of the needed sealing material, the large interior portion is filled with cement grout, and the sealant is confined to the peripheral groove of the joint, as sketched in Section A-A of Fig. 5.

TESTS AND OBSERVATIONS

To obtain information on strength and performance of a panel building of this type, an investigation was undertaken at the National Bureau of Standards. The work is part of a continuing research project in thin-shell precast concrete sponsored by the Navy's Bureau of Yards and Docks. Some phases of these studies have already been completed, while others are still in progress.

Frame strength and behavior

A panel frame of 30-ft span and 4-ft width was assembled for testing its strength and flexural behavior under a gradually increased and uniformly applied roof loading. The framing was designed in accordance with the recommendations set forth in the report of ACI Committee 324,* and the Navy requirements which calls for a prefabricated utility building to withstand 20 psf roof live load and a 70 mph wind. The details of the panels and the jointing are shown in Fig. 3. Since the stresses due to wind are relatively small (causing neither reversal nor uplift under dead load), the assembly was tested only for vertical load. First, a load of 40 psf was placed on the roof, in four equal increments in 2 days. Then, after waiting 24 days, the load was increased to 80 psf in a like manner. This condition was held for 5 days, then the load was removed in a manner similar to the loading procedure. Except for some minor cracking, no significant weaknesses were detected. Also,

*ACI Committee 324, "Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction," ACI JOURNAL, V. 29, No. 11, May 1958 (*Proceedings* V. 54), pp. 921-928.

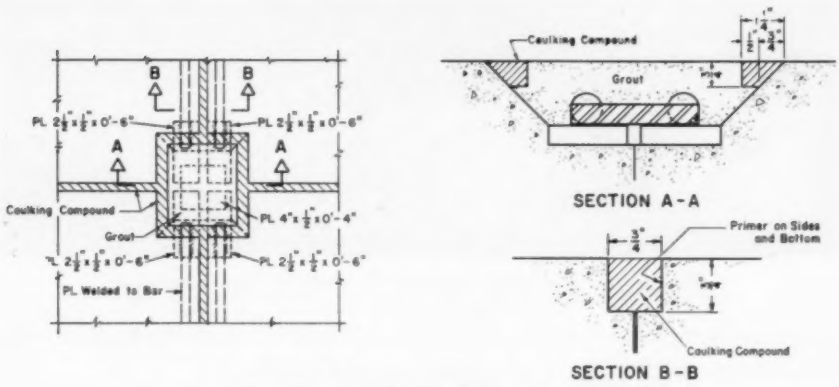

Fig. 5—Typical details for joint seal

flexural and deformational data obtained during the entire test period revealed, in general, a normal behavior anticipated in frame action.

Weathering and sealing tests

A panel building, without the benefit of roofing or siding, presents some maintenance problems which can be more realistically appraised under actual conditions of service. With this view a four-module panel building, of the same design as the strength test frame, complete with end wall enclosure panels, was erected and exposed to weather. For comparative study, the various joints between the panels were sealed with a number of kinds of calking compounds presently available on the commercial market. Most of the sealants were rubber-base compounds of the polysulfide (Thiokol) type, produced by a host of manufacturers, and a few comprised the silicone, neoprene, and epoxy resin group. Observations to date, covering a period of nearly 4 years, have disclosed no leakage through the thin concrete shell of the panels. Of the joint seals, some have failed; while others, representing all basic types, have given a good performance. The building will be kept under surveillance for another 2 years before the adequacy of the various brands of joint sealants can be ascertained.

Fabrication

There is already considerable experience in the production of thin-shell concrete panels. To date, over 10,000,000 sq ft of paneling of this type has been utilized in structural framings, mostly in conjunction with Navy projects. For casting, use has been made of concrete, steel, and plastic molds. Many of the production problems were discussed in a series of papers in the May, 1953, ACI JOURNAL.* In the new application

*ACI JOURNAL, V. 24, No. 9, May 1953 (*Proceedings* V. 49), pp. 773-840.

Fig. 6 — Completed framing of modular panel building

presented here, the framing of the panel, hence the forming and casting, has been simplified by the omission of cross ribs.

Assembly and erection

As yet no standard procedure for the assembly and erection of a panel building has been devised. This is something which must be developed by builders and fabricators geared for mass production and erection of these structures. Many schemes have been considered, including the use of pre-erection modular subassemblies. For handling and erection, the suggested equipment includes simple and mounted derricks, and fork-lift trucks. The latter means was used successfully in the erection of the test building described above.

Further experiments

Design studies have been completed for a panel building of 42-ft span and 4-ft module which will serve as the Navy prototype of a series of structures planned for emergency and mobilization use. The details and arrangement of this wider span framing are similar to those of the 30-ft span presented here. Because of the extent and importance of the contemplated application, it has been deemed desirable to make some further tests and experiments. For this purpose, a new test structure using the increased span is now being built (Fig. 6). A segment of this framing, two panels wide and isolated from the remaining assembly, will be tested for both a vertical and a simulated wind loading. The additional test data thus obtained, as well as supplementary information on problems in fabrication and erection, will be gathered and reported at a future date.

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Title No. 56-63

Effect of Tensile Properties of Reinforcement on the Flexural Characteristics of Beams

By ROBERT G. MATHEY and DAVID WATSTEIN

The effect of magnitude of steel stresses and the nature of the stress-strain characteristics on the center deflection, strain in the concrete, widths and spacing of cracks in the region of constant bending moment, load carrying capacity, and the manner of failure were investigated in a series of flexural beam tests. This series consisted of 12 rectangular concrete beams reinforced with six different types of steel bars with yield strengths ranging from 42,500 to 104,300 psi. The ratio of reinforcement was proportioned inversely to the yield strength, thus providing equal resistance to yielding under tensile forces.

Failure occurred in all the beams at approximately the same load. Test results were compared with values computed with the "straight line" and ultimate strength" theories. The relationship between the computed steel stresses and the center deflections, width and spacing of cracks, and strain in the steel and concrete was determined for the six types of steel bars.

■ THE CURRENT INTEREST IN the properties of reinforced concrete beams containing high-strength deformed steel bars has prompted the National Bureau of Standards to undertake a study of the effect of magnitude of steel stresses on the behavior of flexural members.

The purpose of the investigation was to determine to what extent higher steel stresses affect the resisting moment, width of cracks, strain in the concrete and steel, deflection, load carrying capacity, and manner of failure of beams in which high yield strength reinforcement of smaller cross-sectional area was substituted for intermediate grade deformed bars.

The 12 beam specimens tested in this investigation were of the same size, and contained concrete of the same quality. Six types of deformed bars which differed both in their yield strengths and stress-strain characteristics were used as tensile reinforcement. Two beams were reinforced with intermediate grade bars and were designed to fail by yielding of the steel. Bond and shear failures were precluded by the use of adequate shear reinforcement and anchorage. The ratio of reinforcement was proportioned so that the resistance of the tensile

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David Watstein, chief of the Structural Engineering Section, National Bureau of Standards, has written numerous papers on concrete research and his 1953 ACI Journal paper was awarded the Wason Medal for Most Meritorious Paper. An ACI member since 1941, he is chairman of ACI Committee 208, Bond Stress, and is a member of Commmittees 209, 339, and 716 on the subjects of volume changes, allowable stresses in reinforcement, and high pressure steam curing.

reinforcement to yielding was substantially constant in all beam specimens. This permitted a direct comparison of the beams under a given load.

MATERIALS AND SPECIMENS

Reinforcement

Six types of deformed bars with various stress-strain characteristics were used as longitudinal reinforcement. The nominal yield strengths of these steel bars were 40,000 psi for Type I; 65,000 psi for Types II, III, and IV; and 100,000 psi for Types V and VI. The cross-sectional areas of these bars were determined by the weight-length method. A tensile coupon from each bar size was tested to rupture. Tensile properties of the bars are listed in Table 1, and a typical stress-strain curve for each bar is shown in Fig. 1. The yield strengths of the steel bars varied from the nominal values by + 8.5 percent for Type IV bars to — 1.2 percent for Type V bars. Types I and II bars exhibited a well defined yield strength that was determined by the "halt of the gage" method. Bars III, IV, V, and VI exhibited gradual yielding and their yield strengths were determined by the "0.2 percent offset" method. While Type V bar did not exhibit a sharply defined yield strength, its stress-strain relationship was linear up to a stress of 84,000 psi. Type III bar was specified to exhibit gradual yielding; however, the slope of the stress-strain curve was linear up to a stress of 53,000 psi. Actually there was little difference in the stress-strain properties of Types II and III bars.

Type IV bars exhibited a low modulus of elasticity. The initial tangent moduli of these bars were about 12 percent lower than the moduli of the other five types of bars.

TABLE 1 — PROPERTIES OF REINFORCING BARS

Type of bar	Size of bar	Area, A_s, sq in.	Yield strength, f_y, psi	Tensile strength, f_t, psi	Modulus of elasticity,* E_s, 10^6 psi	Elongation in 8 in., percent
I	#9	0.980	42,500†	78,000	28.5	16.9
II	#7	0.591	69,000†	107,100	28.8	16.0
III	#7	0.597	68,800‡	78,400	29.7	17.2
IV	W8	0.525	69,500‡	83,600	25.7	11.5
IV	W9	0.673	71,200‡	86,100	25.4	10.0
V	#6	0.432	98,800‡	120,600	28.1	6.5
VI	#6	0.433	104,300‡	111,500	28.3	6.9

*The value corresponding to the linear portion of the stress-strain curve. The moduli were determined on bars as received.
†Halt of gage method.
‡0.2 percent offset method.

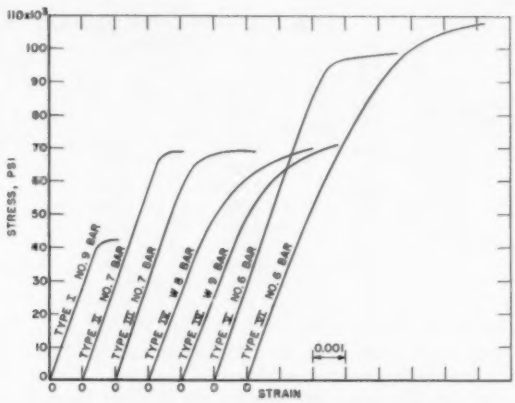

Fig. 1—Stress-strain characteristics of reinforcing bars

All the bars were conventional round reinforcing bars with diamond shaped deformations except Type IV bars. The Type IV bar was a cold twisted, ribbed bar with a dumbbell shaped cross section. Typical reinforcing bars used as tensile reinforcement in the beam specimens are shown in Fig. 2. Height, spacing, and projected length of the deformations were measured on one coupon of each bar size. The properties of deformations appear in Table 2. Deformation of all the reinforcing bars met the requirements of ASTM A 305-53T.

Concrete

The concrete was made from a mix of Type I cement, sand, and gravel proportioned approximately 1:3.2:4.1, by weight. The sand and gravel were

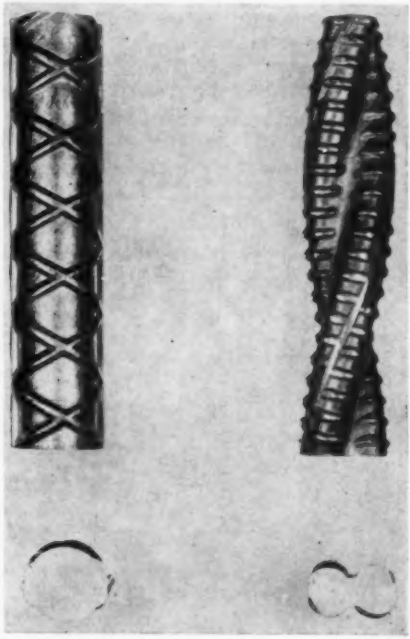

Fig. 2—Typical reinforcing bars. The twisted bar on the right having the dumbbell shaped cross section was designated as the Type IV bar. The other five types of bars had deformations as shown by the bar on the left

TABLE 2 — PROPERTIES OF DEFORMATIONS

Type of steel bar	Bar size	Deformations				
		Width of gap, in.	Average spacing, in.	Average height, in.	Average projected length, in.	Bearing area, sq in. per lineal in.
I	#9	0.088	0.472	0.067	3.14	0.446
II	#7	0.066	0.376	0.056	2.06	0.307
III	#7	0.166	0.341	0.056	2.11	0.346
IV	W8	0.067	0.254	0.042	2.82	0.464
IV	W9	0.088	0.283	0.048	3.26	0.554
V	#6	0.142	0.324	0.048	1.82	0.269
VI	#6	0.160	0.321	0.050	1.82	0.284

siliceous aggregates from White Marsh, Md. The maximum size of the coarse aggregate was 1 in. and the fineness modulus of the sand was 2.71. Gradation of these aggregates is given in Table 3. Water content of the concrete was about 7.7 gal. per sack and the slump ranged from 3 to 6 in. The average compressive strength obtained from tests of control cylinders was 3650 psi and the average values for three cylinders tested for each beam are given in Table 4.

One cylinder of those cast with each specimen was used to determine the stress-strain characteristic of the concrete. The modulus of elasticity, E_c, was the secant value determined by the slope of a line passing through the origin of the stress-strain curve and a point corresponding to one half the compressive strength of the cylinder, 0.5 f_c'. Values of the modulus of elasticity of concrete for each beam specimen are also given in Table 4.

Beam specimens

The specimens were reinforced concrete beams having a cross section of 6 x 15 in. and an effective depth of 12.8 in. The beams were 13 ft long, and were tested over a span of 10 ft. The arrangement of the reinforcement, dimensions of the beams, and the method of loading are illustrated in Fig. 3. The specimens were loaded at the quarter points, and tested as simply supported beams.

The test beams are described in Table 4. The first symbol (Roman numeral from I to VI) designates the type of steel bar used as tensile reinforcement. The last number in the designation denotes the serial number of the beam specimen.

TABLE 3—GRADATION OF AGGREGATES (WHITE MARSH SAND AND GRAVEL)

Sieve size	Cumulative percent retained	
	Sand	Gravel
1 in.	0.0	0.0
3/4 in.	0.0	27.0
1/2 in.	0.0	62.5
3/8 in.	0.0	83.1
1/4 in.	0.0	100.0
No. 4	2.0	100.0
8	17.5	100.0
16	31.5	100.0
30	49.2	100.0
50	76.5	100.0
100	94.0	100.0
Fineness modulus	2.71	8.73

The ratio of longitudinal reinforcement in the beams ranged from a maximum of 2.55 percent for Type I bars having a nominal yield strength of 40,000 psi, to a minimum of 1.12 percent for bars having a yield strength of 100,000 psi. The ratio of reinforcement ranged from 1.54 to 1.56 percent in beams reinforced with bars having a nominal yield strength of 65,000 psi. The total force causing yielding of the tensile reinforcement, $f_y A_s$, as determined by tests of coupons of the reinforcing bars, ranged from 81,600 to 90,300 lb.

Stirrups (#3 bars, 2½ in. on centers) placed in the shear span of the beam and tied to the longitudinal reinforcement were used to safeguard against diagonal tension failures. Steel plates, 4 x 6 x ¼ in., were welded to the ends of the tensile reinforcement to prevent bond failures.

Fabrication and curing of specimens

The stirrups and ties were fabricated in the laboratory and the reinforcement was assembled into a unit before it was placed in the forms. Specimens were cast in steel forms with the tensile reinforcement near the bottom. After 5 days they were removed from the forms and moist cured until 2 days prior to testing at 28 to 30 days. One cylinder was cast from each of the three batches that were used to cast a beam specimen. The three standard 6 x 12-in. cylinders were stored and cured in the same manner as the specimen. The cylinders were tested the same day as the specimen to determine the com-

TABLE 4 — DESCRIPTION OF BEAM SPECIMENS

Beam designation*	Pieces and bar No.	Observed yield strength of reinforcement, f_y, psi	Area of reinforcement, A_s, sq in.	Total yield strength of reinforcement, $A_s f_y$, lb	Maximum load on beam, P_u, lb	Ratio of reinforcement, p	Concrete cylinder strength, f'_c, psi	Modulus of elasticity of concrete, $E_c t$, 10^6 psi	Minimum spacing of cracks, in.	Mode of failure
I-22	2-#9	42,500	1.960	83,300	64,500	0.0255	3520	4.1	4.31	tension
I-23	2-#9	42,500	1.960	83,300	67,500	0.0255	3590	4.4	4.06	tension
II-7	2-#7	69,000	1.182	81,600	63,200	0.0154	3460	4.0	4.14	tension
II-12	2-#7	69,000	1.182	81,600	63,900	0.0154	4200	4.3	4.73	tension
III-8	2-#7	68,800	1.194	82,100	65,100	0.0155	3310	3.6	4.07	tension
III-13	2-#7	68,800	1.194	82,100	64,500	0.0155	4010	4.0	4.80	tension
IV-20	1-W8, 1-W9	70,500	1.198	84,500	61,700	0.0156	3380	3.6	4.01	tension
IV-21	1-W8, 1-W9	70,500	1.198	84,500	61,700	0.0156	3470	4.0	4.38	tension
V-17	2-#6	98,800	0.864	85,400	63,200	0.0112	3810	4.3	4.73	compression
V-19	2-#6	98,800	0.864	85,400	62,100	0.0112	3630	4.0	5.06	uncertain‡
VI-9	2-#6	104,300	0.866	90,300	65,700	0.0113	3730	4.1	4.80	uncertain‡
VI-18	2-#6	104,300	0.866	90,300	63,300	0.0113	3730	3.9	4.65	compression

*The first symbol, Roman numeral from I to VI, designates the type of steel bar used as tensile reinforcement.
†Secant modulus at 0.5 f'_c.
‡Tensile and compressive failures appeared to have occurred simultaneously.

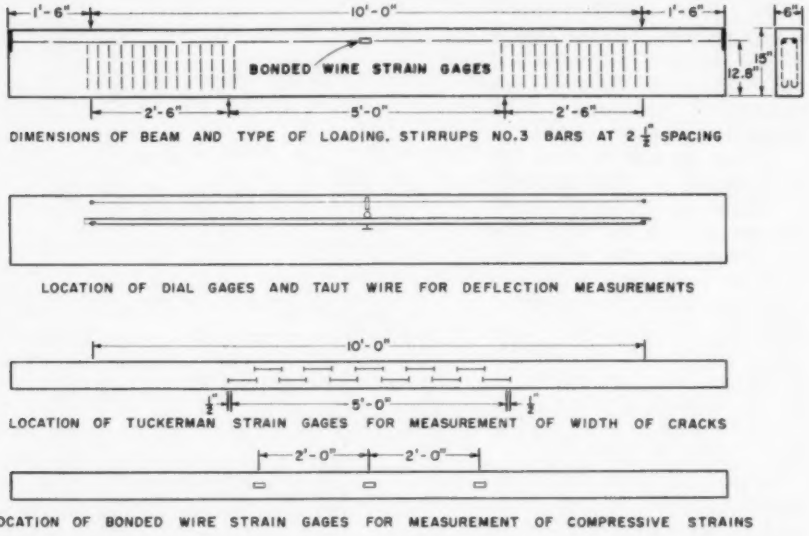

Fig. 3—Details of beams, testing arrangement, and location of strain gages

pressive strength and the modulus of elasticity of the concrete. All concrete was mixed in a tilting drum type mixer of 6 cu ft capacity and placed with the aid of a laboratory internal vibrator.

TESTING PROCEDURE

Test setup

The specimens were tested as simply supported beams over a span of 10 ft in a 600,000 lb capacity hydraulic machine. To facilitate the placement and reading of Tuckerman optical strain gages on the tensile face of the beam, the beams were placed in the testing machine in an inverted position. A sketch of the method of loading and location of Tuckerman strain gages is shown in Fig. 3. Rockers supported the beams at the quarter points and the load was applied through a steel loading beam with attached knife edges. The forces were distributed at the load and reaction points through steel bearing plates 3½ in. wide that covered the entire width of the specimen. The bearing plates at the reaction points were 1 in. thick. The bearing plate was 1½ in. thick under one knife edge and under the other, three ½-in. steel rollers rested on a 1-in. bearing plate.

Instrumentation

The widths of cracks were measured with 11 Tuckerman optical strain gages arranged in two rows at equal distances from the longitudinal center line of the specimen. (Fig. 3). The gages in the two rows were staggered and overlapped 0.5 in. to include every crack which formed in the region of constant bending moment. The Tuckerman gages had a gage length of 6 in. and a 0.5 in. lozenge. The gages were placed on brass bearing strips attached to the surface of concrete with epoxy resin cement. The gages were held firmly in place

with rubber bands attached to brass anchorage disks cemented to the concrete in a similar manner. These gages were equipped with 45-deg prisms to enable observations to be made with the autocollimator in a horizontal position.

The Tuckerman gage readings after formation of a crack were taken to represent the width of crack spanned by the given gage. It was considered impractical to correct the crack width for the tensile strains observed in the concrete prior to formation of cracks within the gage length. In some of the test specimens two or three cracks occurred in the same gage length while no cracks formed in another. Compressive strains were observed in a few cases on the tensile face of the beam where there were no cracks through the gage length, while the adjoining gages spanned cracks. The compressive strain in such cases continued to increase with the tensile stress in the reinforcement. This phenomenon was also observed by Clark[1] and investigated more fully by use of tensile bond specimens.[2]

The average spacing of cracks was determined from the number of cracks and the distance between the two outermost cracks that occurred within the region of constant bending moment.

Deflections at the center of span were measured with 0.001-in. dial gages attached to aluminum bars whose ends were supported at the sides of the beams under the concentrated loads. The dial gages were located as shown in Fig. 3. The gages bore on angle clips attached to the concrete by screws. The aluminum bars with the attached dial gages were removed prior to failure. Deflections were then measured at the center of span by scales attached to the specimen and a taut wire stretched between two points on the side of the beam directly under the concentrated loads.

Strains in the tensile reinforcement at midspan were measured with bonded wire electrical resistance strain gages attached at diametrically opposite points at midplane of the longitudinal bars. The deformations were filed off for a length of about ¾ in. to facilitate the placement of these gages. The gages were Type AB-7, with a gage length of ¼ in. They were attached with epoxy resin cement and waterproofed with Petrosene wax.

The compressive strains in the concrete were also measured with bonded wire electric resistance strain gages. These gages were Type A-1, with a gage length of $1\frac{3}{16}$ in. They were located at midspan and 2 ft either side of midspan along the longitudinal center line of the specimen on the compressive face of concrete (Fig. 3). Gages were attached to concrete with epoxy resin cement.

Test procedure

The loading beam was placed in position before attaching the Tuckerman strain gages to the beam specimen. This procedure was followed to prevent any damage to the Tuckerman gages. The machine load was corrected for the weight of the specimen and loading beam. This correction was equivalent to an applied load at 1720 lb. In the computations this correction factor was added to the machine load.

The load was applied in increments of 2500 lb up to a load of 15,000 lb, and thereafter in increments of 5000 lb to failure. The travel of the head was continued until crushing failure in concrete was observed. Each increment of load was applied at a rate of 4000 lb per min. The following measurements were recorded after each increment of applied load: deflections along the beam, the strains in the reinforcement, the strains in the concrete, and the width of cracks in the region of constant bending moment. In addition to these measurements the location and extent of cracks were recorded immediately after the application of each increment of load.

Observed strains in the steel and concrete were negligible for the first increment of the applied load; therefore, no attempt was made to correct the strain measurements for the initial load. However, the deflections were corrected for the deflection resulting from the weight of the specimen and loading beam. The deflections corresponding to this initial load were computed from the straight line theory. These computed values ranged from 0.0096 in. in beams with Type I bars to 0.0179 in. in beams with Types V and VI bars.

TEST DATA

Mode of failure and load carrying capacity

The beams reinforced with Type I steel bars were designed to fail by yielding of the tensile reinforcement. These beams were used as control specimens. Tensile failures occurred in all the beams except those containing 1.12 and 1.13 percent reinforcement ($f_y = 100{,}000$ psi). Of these, Beams V-17 and VI-18 failed by compression of the concrete, while tensile and compressive failures appeared to have occurred simultaneously in Beams V-19 and VI-9. Typical view of beams after test is shown in Fig. 4.

The load carrying capacity of the beams ranged from 61,700 to 67,500 lb with an average maximum load of 64,000 lb; the deviation of loads at failure from the mean ranged from $+5.5$ to -3.6 percent. The maximum load and mode of failure for the individual beams are given in Table 4.

Deflection of beams

The relationship between the applied load and the observed midspan deflection is shown in Fig. 5. These data along with all other data

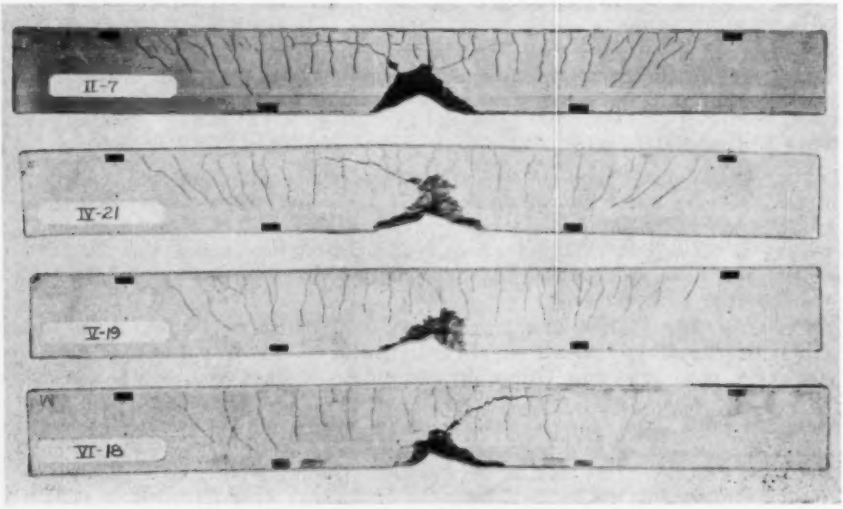

Fig. 4—Typical views of beams after test

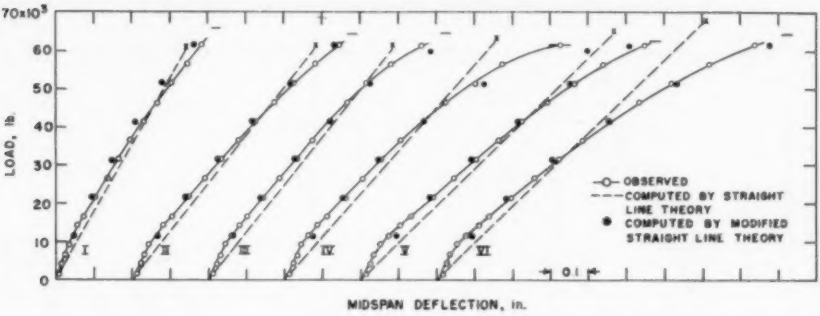

Fig. 5—Load versus observed and computed midspan deflections

presented graphically in the paper represent the average results obtained with two beam specimens. The horizontal lines at the extremity of the observed load-deflection curves represent the maximum loads sustained by the beams. The load-deflection data obtained from duplicate specimens displayed good agreement. The largest difference in midspan deflection of duplicate beams occurred in beams containing 1.13 percent reinforcement (Type VI bars). At a load of 80 percent of the maximum this difference was less than 0.05 in. or 8 percent of the mean midspan deflection. The average difference in duplicate specimens containing the other five types of bars at this load was less than 3 percent of the mean value of midspan deflection. As expected, the deflections increased as the ratio of longitudinal reinforcement was reduced. The deflections of beams with Types II and III bars agreed rather closely. However, the deflections of beams reinforced with Type IV bars departed considerably from the deflections of beams reinforced with bars of Types II and III and having approximately the same ratio of reinforcement. This disparity is explained by the fact that the modulus of elasticity of Type IV bars was 12 percent lower than the moduli of Types II and III bars, and in addition, Type IV bars exhibited gradual yielding to a marked degree. Larger deflections were also observed in beams reinforced with Type VI bars having a curvilinear stress-strain characteristic than in beams containing Type V bars which had a relatively well defined yield strength; these beams had ratios of reinforcement of 1.13 and 1.12, respectively.

The effect of higher steel stress on the midspan deflection is shown more directly in Fig. 6. This graph presents relative deflection values for computed steel stresses equal to 50, 75, and 90 percent of the nominal yield strengths of the bars. The points representing bars having fairly linear stress-strain characteristics are connected by a line faired through the points. For the purpose of direct comparison, the deflections in beams containing 2.55 percent reinforcement (Type I bars) were taken

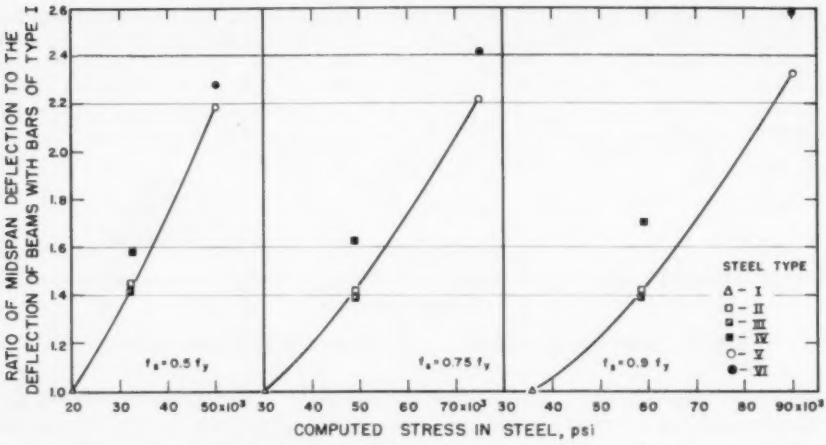

Fig. 6—Comparison of midspan deflections in beams at steel stresses equal to 50, 75, and 90 percent of the nominal yield strengths of the longitudinal reinforcement

as unity. For instance, at stresses equal to one-half the nominal yield strengths of the steels, deflections in beams with ratios of reinforcement of 1.54 to 1.56 percent (Types II, III, and IV bars) ranged from 42 to 58 percent greater than in the control beams. The deflections in beams having ratios of reinforcement of 1.12 and 1.13 percent (Types V and VI bars) ranged from 118 to 128 percent greater than in the control beams.

Stress in reinforcement

The strains in the reinforcement were measured at the center of span. The observed strains were converted into stresses by means of the stress-strain curves shown in Fig. 1. The relationship between the

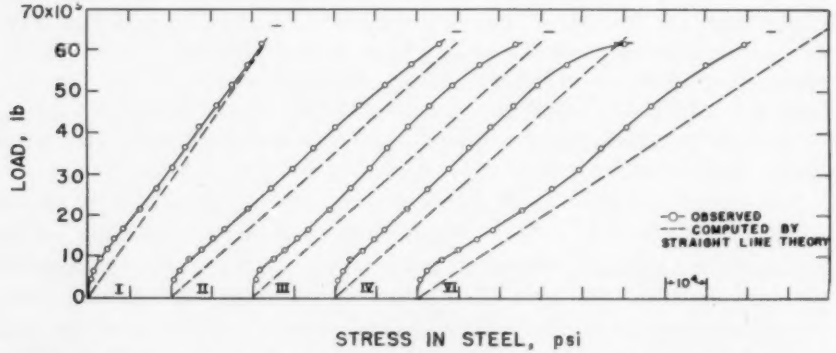

Fig. 7—Load versus observed and computed stresses in the reinforcement at midspan

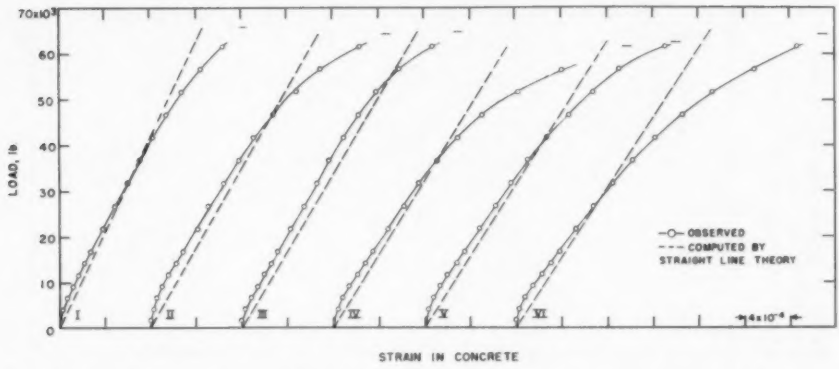

Fig. 8—Load versus observed and computed strains in the concrete. Strains were determined on the compressive face of concrete in the region of constant bending moment

applied load and the observed stress in the reinforcement is shown in Fig. 7. The observed stresses for beams reinforced with Type V steel bars were omitted because of unreliable strain data. The horizontal lines at the extremity of the observed load-stress curves represent the maximum loads sustained by the beams.

Beams reinforced with Types I, II, III, and IV bars failed by yielding of the bars. The maximum observed stresses in these bars approached the yield strengths determined from tensile tests. However, the beams with Type VI bars failed either in compression or simultaneous tension and compression, and the average steel stress observed at a load equal to 95.7 percent of the maximum was only 80 percent of the nominal yield strength.

Concrete strains

The applied load-compressive strain data are presented graphically in Fig. 8. The compressive strains were measured on the surface of concrete and the strain gages were located as shown in Fig. 3.

The maximum difference in compressive strain in the concrete in duplicate specimens was observed in beams containing 1.12 percent reinforcement (Type V bars). At a load of approximately 80 percent of the maximum this difference was equal to 17×10^{-5} in. per in. or 12 percent of the mean value of compressive strain. The average difference in duplicate specimens containing the other five types of bars at this load was less than 6 percent of the mean value of compressive strain. As expected, the compressive strain in the concrete increased as the ratio of longitudinal reinforcement was reduced. The compressive strains in beams with a ratio of reinforcement of 1.56 percent (Type IV bars) were considerably greater than in beams with

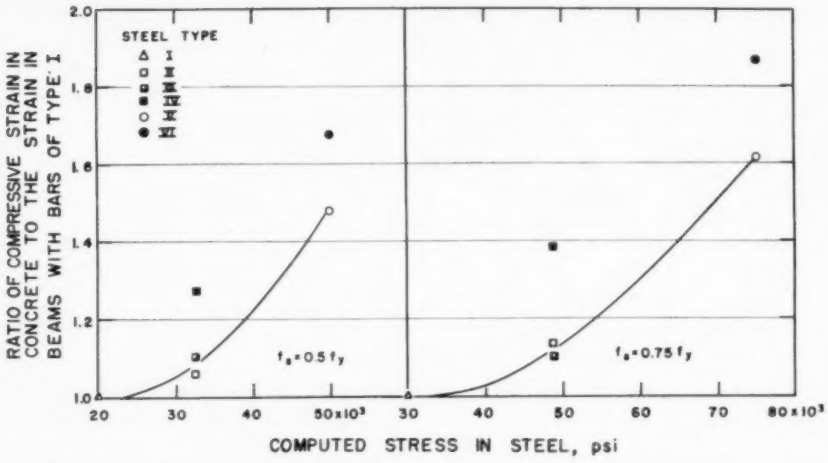

Fig. 9—Comparison of compressive strains in the concrete in beams at steel stresses equal to 50 and 75 percent of the nominal yield strength of the longitudinal reinforcement

bars of Types II and III having approximately the same ratio of reinforcement. This is attributed partly to the lower modulus of elasticity and the gradual yielding characteristic of Type IV bars.

Comparisons of compressive strains in beams with computed steel stresses equal to 50 and 75 percent of the nominal yield strengths are shown in Fig. 9. The curves on this graph connect points representing bars having fairly linear stress-strain characteristics. At stresses equal to one-half the nominal yield strengths of the bars, the compressive strains in beams containing 1.54 to 1.56 percent reinforcement (Types II, III, and IV bars) ranged from 6 to 27 percent greater than in the control beams. In beams containing 1.12 to 1.13 percent reinforcement (Types V and VI bars), the compressive strains ranged from 48 to 67 percent greater than in the control beams. At stresses equal to 75 percent of nominal yield strengths, these increases in compressive strains ranged from 10 to 38 percent for Types II, III, and IV bars, and from 62 to 87 percent for Types V and VI bars.

Widths and spacing of cracks

The relationships between the applied load and the average width of observed cracks is shown in Fig. 10. The cracks were measured on the tensile surface of the beam in the region of constant bending moment (Fig. 3). The largest disparity in the average width of crack in duplicate specimens was observed in beams containing 1.13 percent reinforcement (Type VI bars). At a load of 80 percent of the maximum this disparity was less than 4×10^{-3} in. or 20 percent of the mean

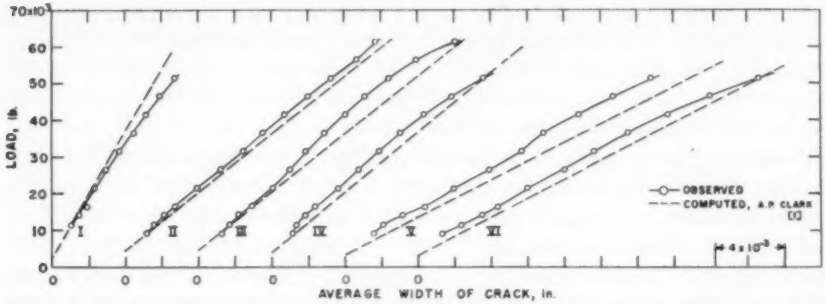

Fig. 10—Load versus observed and computed average width of cracks. Width of cracks were determined on the tensile face of concrete in the region of constant bending moment

value of the average width of crack. As expected, the width of cracks increased markedly as the ratio of longitudinal reinforcement was reduced.

Comparison of the average width of cracks in beams reinforced with high yield strength steels (low ratios of reinforcement) with the width of crack in beams with steel bars of Type I for computed steel stresses equal to 50 and 75 percent of the nominal yield strengths of the bars are shown in Fig. 11. For the purpose of direct comparison the widths of cracks are taken as unity in beams containing 2.55 percent reinforcement (Type I bars). Points representing bars having fairly linear stress-strain characteristics are connected by a curved line. For a stress equal to one-half the nominal yield strengths of the steel in beams with 1.54 to 1.56 percent reinforcement (Types II, III, and IV bars) the average width of cracks ranged from 65 to 77 percent greater than in the control beams. In beams with 1.12 and 1.13 percent reinforce-

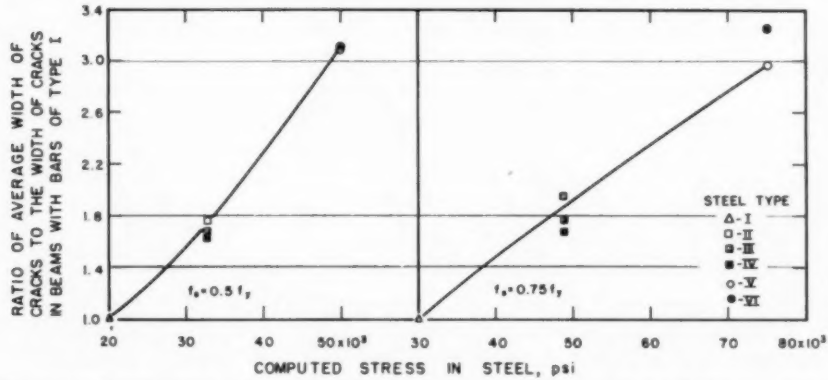

Fig. 11—Comparison of average width of cracks in beams at steel stresses equal to 50 and 75 percent of the nominal yield strength of reinforcement

ment (Types V and VI bars), the width of cracks ranged from 209 to 213 percent greater than in the control beams. At stresses equal to 75 percent of nominal yield strengths, these increases in crack width ranged from 68 to 96 percent for Types II, III, and IV bars, and from 197 to 226 percent for Types V and VI bars.

It can be seen in Fig. 11 that although the curves were faired through points representing bars with well defined yield strengths, the remaining points did not depart appreciably from these curves. The effect of lower modulus of elasticity of Type IV bars on the widths of cracks was less than might have been anticipated. This effect was probably offset by the somewhat smaller average spacing of cracks in beams containing Type IV bars than the spacing in beams with bars of Types II and III, having approximately the same ratio of reinforcement.

The relationship between the computed stress (based on straight line theory) in the reinforcement and observed spacing and width of cracks is shown in Fig. 12.

Some of the gages on each beam specimen spanned more than one crack. No attempt was made to measure the width of individual cracks

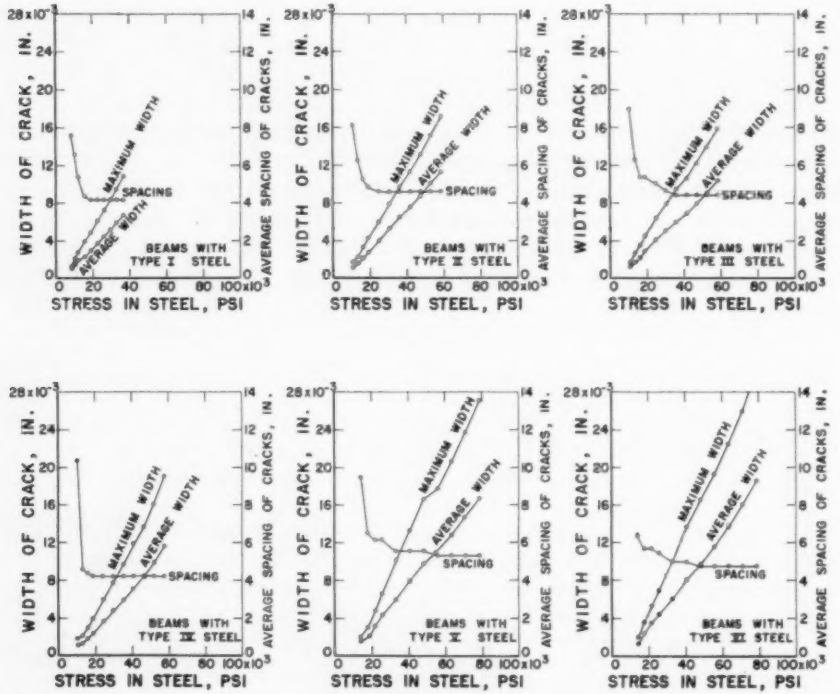
WIDTH OF CRACK, IN.
AVERAGE SPACING OF CRACKS, IN.
STRESS IN STEEL, PSI
MAXIMUM WIDTH
AVERAGE WIDTH
SPACING
BEAMS WITH TYPE I STEEL
BEAMS WITH TYPE II STEEL
BEAMS WITH TYPE III STEEL
BEAMS WITH TYPE IV STEEL
BEAMS WITH TYPE V STEEL
BEAMS WITH TYPE VI STEEL

Fig. 12—Computed stress in the reinforcement versus observed spacing and maximum and average width of cracks

in those gage lengths which had more than one crack. Therefore, it is possible that a few of the cracks having the maximum width may not have been recorded. However, the observed values for the maximum width of crack are presented in Fig. 12. The ratio of the maximum width to the average width of crack was approximately 1.6.

The minimum average spacing of cracks in the control beams was 4.2 in. For beams with 1.54 to 1.56 percent reinforcement ($f_y = 65{,}000$ psi) this minimum spacing of cracks ranged from 4.2 to 4.6 in., and for beams with 1.12 and 1.13 percent reinforcement ($f_y = 100{,}000$ psi) this minimum spacing ranged from 4.7 to 5.3 in. The minimum average spacing of cracks increased with a reduction in the ratio of longitudinal reinforcement. The minimum spacings of cracks for the individual beams are given in Table 4.

In view of the generally greater spacing of cracks observed in beams with reduced ratios of reinforcement (Fig. 12), it would be expected that at a given stress in steel the average width of cracks would be somewhat greater in beams containing the lesser amounts of reinforcement. It is believed that the effect of greater spacing of cracks on the width of cracks was offset by the fact that the reduction of ratio of reinforcement was achieved by using smaller diameter bars. The effective modulus of elasticity of embedded deformed bars is materially increased as the diameter of bars is reduced.[2]

COMPARISON OF OBSERVED AND COMPUTED VALUES

The comparisons of observed and computed values of the midspan deflections, stresses in reinforcement, and the compressive strains in the concrete are shown graphically in Fig. 5, 7, and 8, respectively. The dashed lines in these figures show the computed values determined from the conventional straight line theory. A comparison of the observed and computed average widths of cracks is shown in Fig. 10. The computed values for the average widths of cracks were determined from an expression developed by Clark.[1] A comparison of maximum resisting bending moments computed from the straight line and ultimate strength theories[3] with the observed values is presented in Table 5.

In computing the values of the variables with the straight line theory the observed values of moduli of elasticity of steel and concrete were utilized. The modulus of elasticity of concrete was the secant value determined for 0.5 f_c' while the modulus of steel was taken as the initial tangent value.

The load-deflection curves for beams reinforced with Types I, II, and III bars exhibited fair agreement between observed and computed values (Fig. 5). The disparity between computed and observed deflections was more pronounced for Types IV, V, and VI, particularly

at higher loads. The computed midspan deflections were determined from the expression

$$\Delta = \frac{11\,ML^2}{96\,E_cI}$$

where

M = bending moment in the portion of beam between load points
L = length of beam span
E_c = modulus of elasticity of concrete (secant modulus for 0.5 f_c')
I = moment of inertia of beam cross section transformed to concrete $= \frac{1}{3}\,bk^3d^3 + nA_s\,(1 - k)^2d^2$
n = observed modular ratio
b = width of beam
d = effective depth of beam
kd = depth to neutral axis of transformed section
A_s = area of tension reinforcement

The computed midspan deflections plotted in Fig. 5 are terminated at the calculated yield point of the beam as indicated on the graph by a cross mark. Average maximum observed load is shown in each case by a short dash at the extremity of the observed load-deflection curve.

It was observed that the agreement between computed and observed deflections at higher loads was better for steel bars having a linear stress-strain characteristic than for bars of the same nominal yield strength but exhibiting gradual yielding. The observed deflections of all the beams were less than the calculated values up to a stress of at least one-half the nominal yield strength of the reinforcement. For stresses above this value, the observed deflections in general exceeded the computed deflections. This disparity increased with a reduction in the ratio of reinforcement and was more pronounced with steel exhibiting gradual yielding.

The center deflections were also computed by taking into account the actual values of moduli of elasticity of steel and concrete as they varied during the application of load. The deflections were computed by this method in an effort to obtain better concordance between computed and observed values.

In this method which is designated as the "modified straight line" method, the first step was to compute the steel stress at a given load using the conventional straight line theory. The steel stress was then converted to strain using the actual stress-strain relationship obtained for the reinforcing steel. The compressive strain in the concrete was then computed from this value of the steel strain and the previously computed value of k. The moduli of elasticity for both the concrete

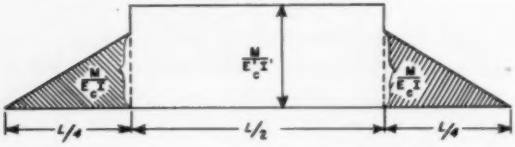

Fig. 13 — Assumed loading of conjugate beam for computing center deflection by "modified straight line" theory

and the steel were next determined as the secant moduli corresponding to these values of strains. The secant modulus of steel was further adjusted by taking into account the contribution of the concrete in resisting tensile stresses after formation of tensile cracks in the concrete. This was accomplished by using the previously observed ratios of the modulus of steel embedded in concrete to the modulus of a freely stressed bar.[2] These values of secant moduli were next used to compute the corresponding values of n' and k' and the corresponding value of the moment of inertia of a cracked section transformed to concrete.

The midspan deflection was then calculated as the moment at the center of a conjugate beam loaded with a M/EI diagram shown in Fig. 13. For the purpose of computation, the E_cI value in the shear spans was assumed to be that given by the conventional straight line theory, since the average stresses in this portion of the beam were relatively low. The expression for the midspan deflection is given by

$$\Delta = \frac{ML^2}{8}\left[\frac{1}{6E_cI} + \frac{3}{4E_c'I'}\right]$$

where E_c' and I' are, respectively, the values of secant

TABLE 5—COMPARISON OF OBSERVED AND COMPUTED MAXIMUM BENDING MOMENTS

Beam	Maximum moments													
	Observed M_o, in.-lb	Straight line theory M_s,* in.-lb	Straight line theory M_s,† in.-lb	Ultimate strength theory M_u,* in.-lb	Ultimate strength theory M_u,† in.-lb	Ultimate strength theory M_u,‡ in.-lb	$\frac{M_o}{M_s}$*	$\frac{M_o}{M_s}$†	$\frac{M_o}{M_s}$‡	$\frac{M_o}{M_u}$*	$\frac{M_o}{M_u}$†	$\frac{M_o}{M_u}$‡
I-22	968,000	908,000	906,000	872,000	870,000	870,000	1.07	1.07	1.07	1.11	1.11	1.11
I-23	1,012,000	912,000	910,000	876,000	874,000	874,000	1.11	1.11	1.11	1.16	1.16	1.16
II-7	978,000	915,000	917,000	855,000	857,000	857,000	1.07	1.07	1.07	1.14	1.14	1.14
II-12	958,000	921,000	924,000	888,000	891,000	891,000	1.04	1.04	1.04	1.08	1.08	1.08
III-8	976,000	913,000	916,000	915,000	849,000	851,000	850,000	1.07	1.07	1.07	1.15	1.15	1.15
III-13	968,000	919,000	922,000	921,000	864,000	886,000	885,000	1.05	1.05	1.05	1.10	1.09	1.09
IV-20	926,000	947,000	920,000	883,000	874,000	855,000	828,000	0.98	1.01	1.05	1.06	1.08	1.12
IV-21	926,000	952,000	925,000	888,000	879,000	860,000	833,000	0.97	1.00	1.04	1.05	1.08	1.11
V-17§	948,000	978,000	972,000	966,000	902,000	898,000	893,000	0.97	0.98	0.98	1.05	1.06	1.06
V-19	932,000	974,000	969,000	957,000	892,000	889,000	884,000	0.96	0.96	0.97	1.04	1.05	1.05
VI-9	986,000	1,031,000	997,000	925,000	943,000	918,000	865,000	0.96	0.99	1.07	1.05	1.07	1.14
VI-18§	950,000	1,029,000	994,000	923,000	943,000	918,000	865,000	0.92	0.96	1.03	1.01	1.03	1.10

*Using f_y determined for 0.20 percent offset.
†Using f_y determined for 0.15 percent offset.
‡Using f_y determined for 0.10 percent offset.
§These specimens failed in compression.

modulus of concrete and the corresponding moment of inertia of a cracked section transformed to concrete.

The midspan deflections determined by the modified straight line method agreed better with the observed deflections than the values based on the conventional straight line theory. The observed and computed deflections are shown in Fig. 5.

Comparison of the computed and observed stresses in the reinforcement is shown in Fig. 7. Steel stress data from beams reinforced with Type V bars were omitted from this figure because of unreliable strain gage readings. The observed strains were converted into stresses by means of the stress-strain curves shown in Fig. 1.

The observed and computed values of steel stresses show good agreement for beams reinforced with Type I bars, particularly at the higher stresses. However, in beams reinforced with the higher yield strength bars (lower ratios of reinforcement), the observed stress in the steel bars is considerably less than the computed stress. This is particularly true in beams reinforced with Type VI bars. The straight line theory thus predicted stresses that were in excess of observed stresses in all cases. The difference between the observed and computed stresses increased as the ratio of reinforcement decreased. This difference between the computed and observed stress in the reinforcement can be attributed partly to the basic assumption incorporated in the straight line theory in which the tensile stress in the concrete is neglected. This relation between observed and computed stresses in the reinforcement agrees with the work of other investigators.[4]

The computed and observed strains in the concrete are compared in Fig. 8. The computed strain was determined from the compressive stress at the surface of concrete calculated with the straight line theory and the observed modulus of elasticity of the concrete. The values of the observed compressive strains in the concrete in all cases were less than the computed values up to the load equal to approximately one-half of the ultimate load on the beam. At higher loads, the observed strains in concrete exceeded the computed values.

As in the case of load-deflection relationship, the departure of the observed compressive strains from the computed values at high steel stresses was greater for bar Types IV, V, and VI, than for Types I, II, and III. The departure of observed strains from computed values was particularly marked for Types IV and VI whose stress-strain characteristics showed the greatest departure from linearity. The modulus of elasticity of Type IV bars was 12 percent less than that of a conventional bar at a stress of 30,000 psi, while at a stress of 60,000 psi this difference increased to 28 percent. Similarly, for Type VI bars, the modulus at stresses of 50,000 and 95,000 psi was, respectively, about

13 and 27 percent less than that of Type V bars with a fairly well defined yield strength.

The yield moments computed from the straight line theory and the maximum moments computed from the ultimate strength theory are compared in Table 5 with the observed moments. The yield moments computed by the straight line theory were based on yield strengths determined from 0.20, 0.15, and 0.10 percent offsets, while the moduli of elasticity E_s and E_c were taken as the initial tangent modulus of steel and secant modulus of concrete at 0.5 f_c', respectively. These three sets of yield strength values were also used in computing the maximum resisting moments with the ultimate strength theory. In all three cases the agreement between the observed ultimate and computed maximum moment was better in general for the straight line theory than for the ultimate strength theory. It was found that the yield moment based on straight line theory could be predicted with somewhat greater accuracy for this series of beams if the 0.15 percent offset method were used to determine the yield strength of the steel bars. However, for the ultimate strength theory the agreement between the computed maximum moments and the observed values was slightly better when the 0.20 percent offset was used to determine the yield strength of the bars.

Other investigators[5] also reported good agreement between the observed and computed (with straight line theory) maximum resisting bending moments for beams adequately reinforced against diagonal tension failures. They also found that the maximum bending moment was determined by the total yield strength of the reinforcement and not by the type of bars.

The observed and computed values of the average width of cracks are compared in Fig. 10. Each curve represents the results obtained from two beam specimens. In the application of Clark's[1] formula for the average width of cracks, bars of Type IV steel were assumed to be round and their diameters were determined from their cross-sectional areas. The average width of cracks was computed from the following expression:

$$w = 2.27 \times 10^{-6} \left(\frac{h-d}{d}\right) \frac{D}{p} \left[f_s - 56.6 \left(\frac{1}{p} + n\right) \right]$$

where

w = average width of cracks, in.
D = diameter of reinforcing bar, in.
f_s = computed stress in reinforcement, psi
h = over-all depth of beam, in.
p = ratio of longitudinal reinforcement

The other terms in the equation were previously defined in the paper.

Comparison of the observed and computed crack widths in Fig. 10 indicates that the mean crack width was predicted with reasonable

accuracy for beams reinforced with the six different types of steel bars. The shape of the stress-strain curve of the bars did not appear to have a consistent effect on the relationship between the mean crack width and the computed stress in the reinforcement.

It must be pointed out that the reasonably good agreement between the observed and computed widths of cracks was obtained even though Clark's formula was based on results obtained with bars of intermediate grade, whereas the data on width of cracks presented in this paper represent steel stresses ranging up to about 75,000 psi.

SUMMARY

The beams in this study contained reinforcement of six different types with nominal yield strengths ranging from 40,000 to 100,000 psi. The ratio of reinforcement was proportioned inversely to the yield strength, thus providing equal resistance to yielding under tensile forces. This was accomplished by substituting higher yield strength bars of smaller diameter, bar for bar.

It was found that the load carrying capacity of the beams was not appreciably affected by the tensile properties of the longitudinal reinforcement. The control beams were designed to fail by yielding of the reinforcement and contained 2.55 percent reinforcement (Type I bars). Beams with 1.12 and 1.13 percent reinforcement (f_y = 100,000 psi) failed in compression or simultaneously in compression and tension while beams containing 1.54 to 1.56 percent reinforcement (f_y = 65,000 psi) failed in tension.

At a given load the midspan deflections, strains in the concrete, and the average width of cracks in the region of constant bending moment were greater in beams with lower ratios of reinforcement and corresponding higher steel stresses. Beams containing steel bars exhibiting gradual yielding and low modulus of elasticity developed larger compressive strains and center deflections than corresponding beams reinforced with bars having linear stress-strain characteristics and containing approximately the same ratios of reinforcement. However, the gradual yielding and low modulus properties of the tensile reinforcement had no similar effect on the relationship between the average width of cracks and the computed stress in the reinforcement.

Observed midspan deflections and compressive strains in the concrete in the region of constant moment were less than computed values (straight line theory) up to a stress of appproximately one-half the yield strength of reinforcement. At higher steel stresses, the observed values of concrete strain and deflection in general exceeded the computed values. Better concordance between computed and observed deflections was obtained by a method (modified straight line method) that took into account the actual values of the moduli of elasticity of

steel and concrete as they varied during the application of load. The mean width of cracks was predicted by Clark's[1] formula with reasonable accuracy for all the beam specimens.

The straight line theory predicted stresses in the reinforcement in excess of observed stresses in all cases; the disparity between observed and computed stresses increased with a decrease in the ratio of reinforcement. Yield moments computed from the straight line theory agreed in general better with observed values than moments computed from the ultimate strength theory. The yield moments were predicted with somewhat greater accuracy from the straight line theory when the yield strengths of the reinforcement were taken as determined from 0.15 percent offset, rather than the 0.20 percent offset normally used.

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Title No. 56-64

Carbon Dioxide in Hydrated Portland Cement

By W. F. COLE and B. KROONE

A study is made of the way in which carbon dioxide is held in carbonated samples of portland cement mortar and of calcium silicate hydrate using differential thermal, thermogravimetric, and x-ray diffraction methods. The results indicate that the carbon dioxide is chemically bound as calcium carbonate, largely in the form of poorly crystallized vaterite, aragonite, and calcite, and not as a silicate mineral. The carbonate minerals are intimately associated with the siliceous residue that results from the carbonation of the hydrated cement minerals. They react with this residue when the samples are heated to yield the unstable larnite (β-$2CaO \cdot SiO_2$) which, at room temperature, is discernible in a rapidly cooled sample. The temperature of the reaction is well below that at which well-crystallized calcite and quartz react to produce wollastonite ($CaO \cdot SiO_2$). Some conclusions are drawn as to the mechanism by which the carbonation of cement minerals could take place.

■ ALTHOUGH THE PHYSICAL AND CHEMICAL EFFECTS of the carbonation of cements and mortars have been studied by many workers,[1-12] there has as yet been no detailed study aimed at establishing how the carbon dioxide is held in the cements and mortars. Most workers have assumed that it exists in the form of calcite but in work at this Division[12] (some of it unpublished) the quantity of calcite detected by x-ray diffraction and differential thermal methods is always far less than the amount of carbon dioxide recoverable from the samples. Gaze and Robertson[13] have observed similar results from samples of tobermorite [calcium silicate hydrate (1)]. To study the matter further, Kroone and Blakey[12] have determined the amount of carbon dioxide and water vapor freed from carbonated mortars at different temperatures. They have found that although large amounts of carbon dioxide can be taken up by mortars to form calcite, a considerable amount is also taken up in some less stable form by a mechanism as yet undetermined. In the present paper some of the mortars used by Kroone and Blakey and a carbonated sample of calcium silicate hydrate are discussed in detail following an examination by differential thermal, thermogravimetric, and x-ray diffraction methods. As a result, some of the earlier obscurities in the mechanism of the carbonation of cement minerals are resolved.

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Dr. Cole, officer-in-charge of mineralogical and crystallographic investigations, graduated in physics and geology from the University of Western Australia. His post graduate work at the university concerned x-ray studies of clay minerals. He received his Ph.D. from the Cavendish Laboratory, Cambridge University, for further x-ray studies of feldspar minerals. Since being with CSIRO he has continued to use x-rays to study clay minerals, sorel cement, portland cement, and other building materials.

Mr. Kroone, experimental officer of the concrete and testing laboratory, graduated doctorandus in chemistry from the University of Amsterdam. Prior to joining the CSIRO research group in 1956, Mr. Kroone was engaged in teaching and in the gas industry.

MATERIALS EXAMINED

Details of the samples examined are set out in Table 1. The mortars had a sand-cement ratio of 4.0 and a water-cement ratio of 0.60 by weight. All were made from the same batch of sand and the same brand of portland cement (chemical analysis: CaO, 64.7 percent; SiO_2, 20.6 percent; Al_2O_3, 5.3 percent; Fe_2O_3, 3.3 percent; MgO, 0.6 percent; SO_3, 1.9 percent; Na_2O, 0.7 percent; K_2O, 0.7 percent; loss on ignition, 0.9 percent. Compound composition: $3CaO \cdot SiO_2$, 58.6 percent; $2CaO \cdot SiO_2$, 14.8 percent; $3CaO \cdot Al_2O_3$, 8.2 percent; $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$, 10.2 percent; $CaSO_4$, 3.2 percent; free CaO, 0.6 percent). Samples were chosen so that comparisons could be made between the amounts and character of the carbon dioxide developed under very different conditions. Thus Samples B14, B13V, and C6 were cured under conditions (either very damp or very dry) unfavorable for carbonation, whereas B6, B8V, and C7 after being exposed to noncarbonating conditions were finally cured under conditions favorable for carbonation. This was done with dry carbon dioxide, since the samples contained sufficient water lost below 108 C (Table 1) for reaction to take place. Sample A3V, on the other hand, was exposed for 3 and 30 months to the atmosphere in a room maintained at 65 percent relative humidity and 21 C. The sample of calcium silicate hydrate was made by pressure molding a mixture of lime, silica, and water in the proportions by weight of 0.5:1.0:0.1 and autoclaving the product for 7 hr in saturated steam at 220 C.* The sample was carbonated for 4 days in an atmosphere of damp carbon dioxide. In some experiments free lime, obtained by calcining calcium carbonate of A.R. purity, was carbonated in air.

EXPERIMENTAL PROCEDURES

Differential thermal analyses were made with automatic equipment described by Carthew and Cole.[14] A heating rate of 10 C per min was used with about 2 g of material and a recording sensitivity of 2.2 μV per mm deflection of the recording pen. When samples were removed from the equipment for x-ray examination a second run was made and at the required temperature the furnace was quickly raised, a portion of the sample being taken from the top of the sample holder. The furnace was then lowered and the remainder of the sample taken to a higher temperature for further sampling.

Thermogravimetric measurements were made with a Chevenard automatic thermobalance, using a heating rate of 10 C per min and a recording sensitivity of 2.38 mg per mm deflection on the photographic film. The dehydration curves were analyzed by plotting the first derivative graphically.[15] Combined differential

*Taylor, W. H., and Moorehead, D. R., Unpublished data, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

TABLE I—DATA RELATING TO WATER CONTENT AND CARBON DIOXIDE CONTENT OF SAMPLES EXAMINED*

Material	Specimen No.	Treatment	H_2O below 108 C, percent	H_2O above 108 C, percent	Total H_2O, percent	CO_2 below 500 C, percent	CO_2 above 500 C, percent	CO_2 combined with alkalies, percent	Total CO_2, percent	Total CO_2+H_2O, percent
Portland cement mortars	B14	27 days air 100 percent relative humidity 21 C	3.45	3.13	6.58	0.0	1.47	0.16	1.63	8.13
	B6	27 days air 100 percent relative humidity 21 C, 28 days dry CO_2	2.32	2.43	4.75	1.46	4.07	0.14	5.67	10.42
	B13V†	27 days air 100 percent relative humidity 21 C	2.70	3.12	5.82	0.0	1.48	0.17	1.65	7.47
	B8V†	27 days air 100 percent relative humidity 21 C, 28 days dry CO_2	2.19	1.50	3.69	2.32	5.05	0.13	7.50	11.19
	C6	2 days air 100 percent relative humidity 21 C, 18 hr 256 C, 32 days dry CO_2	0.46	1.51	1.97	0.0	1.26	0.16	1.42	3.39
	C7	2 days air 100 percent relative humidity 21 C, 18 hr 256 C, 32 days air 65 percent relative humidity 21 C	0.91	1.06	1.97	1.14	1.75	0.14	3.03	5.00
	A3V	3 months air 65 percent relative humidity 21 C	0.89	2.08	2.97	1.81	2.50	0.14	4.45	7.42
	A3V	30 months air 65 percent relative humidity 21 C	1.09	1.13	2.22	1.92	3.96	0.15	6.03	8.25
Calcium	P76	As autoclaved	1.72	5.92	7.64	0.35	0.72		1.07	8.71
silicate hydrate	P77	4 days damp CO_2	3.81	2.22	6.03	12.16	2.54		14.70	20.73

*Based on Table 2 of Reference 12. After treatment the samples were brought into equilibrium with a room held at 21 C and 50 percent relative humidity before analysis. The figures are quoted on this basis.
†Specimen vibrated during molding.

thermal analyses and thermogravimetric measurements were made with the equipment described by Powell.[10]

X-ray examinations were made with monochromatic copper radiation using a Guinier-type focussing camera of 114.6 mm diameter. This camera is of high dispersion and enables small differences in spacings to be observed and complex mixtures to be analyzed with much greater certainty than with conventional cameras. Samples were examined in their natural state and after heating to different temperatures in the differential thermal equipment. After heating they were sealed in "durex" packets so as to prevent recarbonation during the x-ray exposure.

One sample (B6) was fractionated into two parts at a specific gravity of 2.59, using a mixture of bromoform and carbon tetrachloride and a laboratory centrifuge. A greater proportion of carbonated material was retained in the light fraction.

The data relating to the water content and carbon dioxide content of the samples as set out in Table 1 were obtained as described by Kroone and Blakey.[12]

RESULTS

Differential thermal analysis

The differential thermal curves of the samples of mortar, of calcium silicate hydrate, and reference curves for calcite and calcium hydroxide are illustrated in Fig. 1. Those for mortar samples cured under non-carbonating conditions (A, D, and F, Fig. 1) are similar. All show

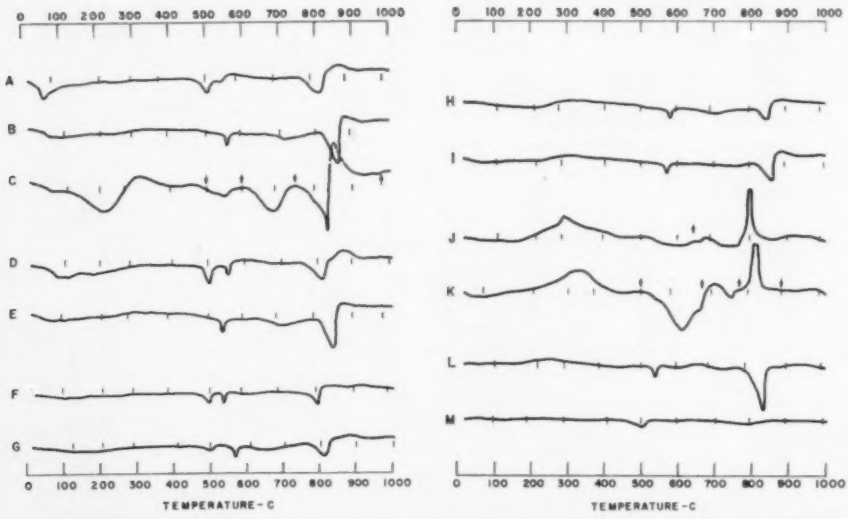


Fig. 1—Differential thermal curves of samples of mortar, of calcium silicate hydrate, and reference curves of calcite and calcium hydroxide. Points above the curves at which samples have been taken for x-ray examination are indicated. Curve A, Sample B14; B, B6 carbonated; C, B6 carbonated and fractionated (light fraction); D, B13V; E, B8V carbonated; F, C6; G, C7 carbonated; H, A3V carbonated 3 months; I, A3V carbonated 30 months; J, P76; K, P77 carbonated; L, 5 percent $CaCO_3$; M, 5 percent $Ca(OH)_2$. See also Table 2

a low-temperature endothermic peak, due probably to the dehydration of interlayer water from hydrated cement minerals, two small endothermic peaks at about 500 C and 550 C due to $Ca(OH)_2$ dehydration and α-β quartz inversion respectively (L and M, Fig. 1), a weak endothermic peak between 600-750 C and a stronger one at about 800 C due to the decomposition of $CaCO_3$ as well-crystallized calcite, followed by a weak exothermic reaction. The curves for the mortar samples cured under carbonating conditions (B, E, G, H, and I, Fig. 1) differ little from those for the noncarbonated mortars just discussed. The endothermic peak due to $Ca(OH)_2$ is reduced in intensity or eliminated, the endotherm between 600-750 C and that at about 800 C are slightly increased in intensity, and the final exothermic reaction is sharper.

An estimate has been made of the amount of well-crystallized calcite in the mortar samples as determined by differential thermal analysis, using as a standard a mixture of 5 percent $CaCO_3$ (well-crystallized calcite) and 95 percent silica (quartz). The results are compared in Table 2 with figures obtained by assuming that all the carbon dioxide found by chemical analysis (see Table 1) is combined as calcite. They show that only a small part of the increased carbon dioxide content of the carbonated mortars can be due to well-crystallized calcite. The

TABLE 2—$CaCO_3$ AND CALCITE CONTENTS OF SAMPLES OF MORTAR AND OF CALCIUM SILICATE HYDRATE

Material	Sample, (see Fig. 1 for corresponding curves)	Chemical analysis	Differential thermal analysis		
				Calcite	
		$CaCO_3$ total possible,* percent	$CaCO_3$ from $Ca(OH)_2$,† percent	Well crystallized,‡ percent	Poorly crystallized,§ area in arbitrary units
Portland cement mortars	B14 noncarbonated, Curve A	3.7	—	2.5	4
	B6 carbonated, Curve B	12.8	5.1	3.8	6
	B13V noncarbonated, Curve D	3.7	—	2.4	3
	B8V carbonated, Curve E	17.0	4.3	3.3	6
	C6 noncarbonated, Curve F	3.2	2.6	1.2	2
	C7 carbonated, Curve G	6.9	4.0	1.5	5
	A3V carbonated 3 months, Curve H	10.1	—	1.0	5.5
	A3V carbonated 30 months, Curve I	13.7	—	1.9	3
Calcium silicate hydrate	P76 noncarbonated, Curve J	2.4	—	4.7	26
	P77 carbonated, Curve K	33 3	—	5.3	115

*Based on the total CO_2 content recorded in Table 1.

†Based on the area of the endothermic peak at about 500 C for the noncarbonated sample of the pair.

‡Based on the area of the endothermic peak at about 820 C.

§Area of endothermic peak between 600-750 C.

only difference of any significance between the differential thermal curves of the carbonated and noncarbonated mortars is the increase in the size of the shallow endothermic peak between 600-750 C in the carbonated samples. An attempt has been made to measure the area of this peak and the results are set out in Table 2. The increase in the size of the peak in the carbonated samples only appears sufficient to account for a small part of the increased carbon dioxide content of the samples. It must then be deduced that the carbon dioxide in carbonated mortars is loosely held and freed slowly upon heating, with little to no endothermic effect.

It should be noted that for Sample A3V, exposed to the atmosphere for periods of 3 and 30 months, the differential thermal curves are practically identical (H and I, Fig. 1). However, the longer exposure has resulted in a slight increase in the total amount of $CaCO_3$ and in the amount of well-crystallized calcite (Table 2).

Interesting results are obtained by separating Sample B6 in a liquid of specific gravity 2.59 and examining the light fraction by differential thermal analysis. If the curve of this material (C, Fig. 1) is compared with that of the nonfractionated sample (B, Fig. 1) it is seen that all thermal effects are accentuated but that there is a greater increase in the size of the endothermic peak between 600-750 C as compared with the one at 800 C (see Table 2). Also, additional weak endothermic effects occur at about 400 C and between 500-600 C (even allowing for the α-β quartz inversion at about 550 C). Gravity separation has thus increased the concentration of some weak thermally reactive component or components in the light fraction.

Results of even greater significance are obtained from the differential thermal curves of the samples of calcium silicate hydrate. That of the noncarbonated material (J, Fig. 1) shows, apart from an intense exothermic reaction at 800 C, only weak thermal reactions. The curve is typical of that to be expected from tobermorite.[13,17,18] The endothermic peak between 500-700 C and part of that between 700-800 C are not due to tobermorite; the one at the higher temperature indicates the presence of some well-crystallized calcite. The curve of the carbonated sample (K, Fig. 1), although generally similar to that of the noncarbonated sample, shows the development of a strong endothermic reaction from between 500-700 C. The first inflection in this endotherm is due to the α-β inversion of quartz, the second is of unknown origin but both are present in the curve for the original material. The endothermic peak between 700-800 C is modified in shape but unaffected in area through carbonation. The amounts of well-crystallized calcite in the samples of calcium silicate hydrate before and after carbonation, as estimated from the differential thermal curves, are compared in Table 2 with figures obtained by assuming that all the carbon dioxide

found by chemical analysis (see Table 1) is combined as calcite. Thus, as in the case of the mortars the increased carbon dioxide content of the carbonated sample cannot be due to well-crystallized calcite. However, a comparison of the size of the endothermic peak between 500-700 C for the samples of calcium silicate hydrate before and after carbonation indicates that this peak *must* be associated with the carbon dioxide taken up by the sample. It thus becomes reasonable to assume that this also applies to the endothermic peak between 600-750 C for the gravity-separated mortar Sample B6, and probably also to the mortar samples generally.

Thermogravimetric analysis

Curves showing the decomposition on heating of mortar Samples B14 (noncarbonated sample of B6), B6 (carbonated), a separated fraction of B6, and B8V (carbonated) are illustrated in Fig. 2. Those of the noncarbonated (P76) and carbonated (P77) samples of the calcium silicate hydrate are illustrated in Fig. 3. In all cases the loss in weight with temperature is continuous until a temperature of about 800 C

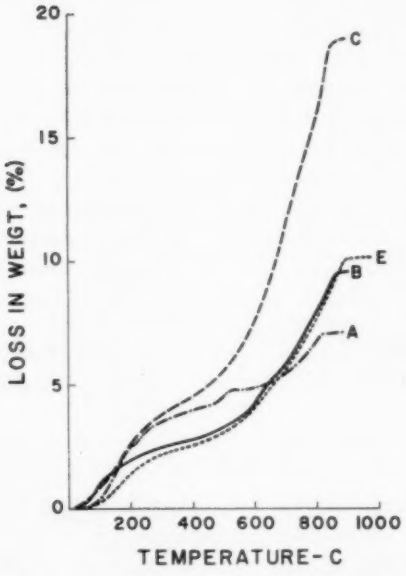


Fig. 2—Weight-loss curves of mortar samples as determined on a Chevenard automatic thermobalance. Curve A, Sample B14; B, B6 carbonated; C, B6 carbonated and fractionated (light fraction); E, B8V carbonated

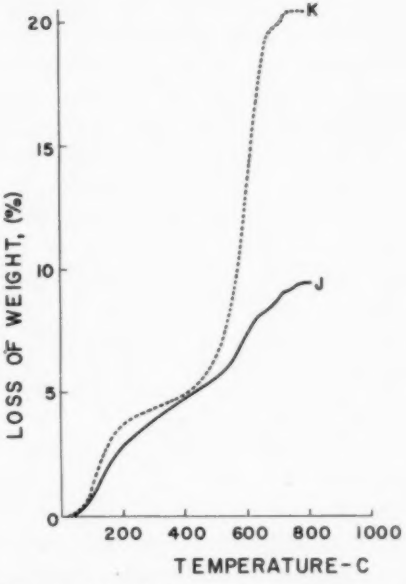


Fig. 3—Weight-loss curves of samples of calcium silicate hydrate as determined on a Chevenard automatic thermobalance. Curve J, Sample P76; K, P77 carbonated

is reached, where decomposition is complete. In the mortar Samples B14 (A, Fig. 2) and B6 (B, Fig. 2) carbonation has reduced the weight lost in the temperature range 200-600 C but the total weight loss is increased. This is probably due to the fact that Sample B14 (noncarbonated) possesses hydrated cement minerals which lose water at a low temperature, and that in the carbonation of these minerals this water is replaced by carbon dioxide (Table 1 and Kroone and Blakey[12]). Curve C, Fig. 2, shows clearly that gravity fractionation of the mortar Sample B6 has markedly increased the amount of material in the light fraction that can be decomposed by heat. The decomposition curves of the samples of calcium silicate hydrate show that carbonation has produced effects on heating similar to those for the mortar samples.

Further information on the decomposition of the samples on heating is obtained by plotting the differential of the weight loss, as has been done in Fig. 4. These curves are, apart from exothermic phenomena, similar to the differential thermal curves of Fig. 1, and they have been assigned the same reference letter to allow ease of comparison. Above each differential weight-loss curves, the loss in weight in the mortar samples at temperatures of 350 C, about 775 C, 800 C, and 1000 C are indicated, as well as the difference in these values. The loss in weight and difference in loss for the samples of calcium silicate hydrate are similarly indicated at temperatures of 350 C, about 680 C, and 800 C.

Since the curves of the samples of calcium silicate hydrate are simpler than those of the mortar samples they are described first for convenience. Up to a temperature of 350 C the curves of the two samples (J and K, Fig. 4) are similar, but from 350 C to about 680 C the carbonated sample loses considerably more weight than the noncarbonated sample. However, beyond 680 C the loss in weight of both samples is small and about the same for each sample. This last weight loss is due to the decomposition of well-crystallized calcite, and the amounts present are in general agreement with those found by chemical analysis (CO_2 lost above 500 C, Table 1) and differential thermal analysis (Table 2). Clearly the bulk of the carbon dioxide cannot be present as well-crystallized calcite.

In the mortar Samples B6 (B, Fig. 4) and B14 (A, Fig. 4) the initial part of the curves, up to a temperature of 350 C, are similar but beyond this temperature they differ considerably. The noncarbonated Sample B14 (A, Fig. 4) shows the presence of calcium hydroxide by a peak at about 500 C. This peak is not present in the carbonated Sample B6 (B, Fig. 4) but carbonation introduces an additional peak at a temperature of about 610 C. This temperature is nearly the same as that of the main peak in the curve of the carbonated sample of calcium silicate hydrate (K, Fig. 4). From about 600-800 C the loss in weight of all the mortars is continuous and it is not possible to decide from the

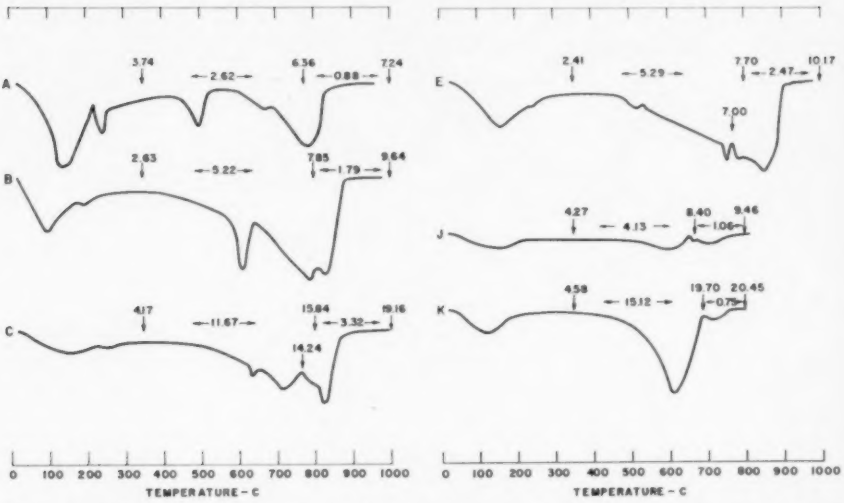


Fig. 4—Differential plot of weight-loss curves of samples of mortar and of calcium silicate hydrate. Above each curve are indicated the loss in weight (percent) at certain temperatures and the difference in weight loss (percent) between these temperatures. Curve A, Sample B14; B, B6 carbonated; C, B6 carbonated and fractionated (light fraction); E, B8V carbonated; J, P76; K, P77 carbonated

differential weight-loss curve at what temperature well-crystallized calcite starts to decompose. With reference to the curves shown in Fig. 4 this temperature has been taken as 800 C for the carbonated samples and 775 C for the noncarbonated mortar samples (based on the differential thermal curves of Fig. 1). The amount of well-crystallized calcite present, on this basis, is clearly small (Fig. 4), indicating that most of the carbon dioxide must be less firmly bound. Of the differential weight-loss curves of the mortar samples illustrated in Fig. 4, that of the light fraction of Sample B6 (Curve C) is the only one which closely resembles in its entirety the corresponding differential thermal curve (C, Fig. 1). In particular there is an appreciable weight loss at about 700 C corresponding to the endothermic peak at this temperature.

X-ray analysis

Unheated samples — The x-ray powder patterns of the noncarbonated mortar samples show the presence of much quartz, some calcium hydroxide, and some calcite, with quite a few weak and very weak lines unaccounted for (see Table 3 for results on Sample B13V). In general, it is difficult to recognize hydrated cement minerals in set mortars, particularly after short curing times, but the additional lines could be

TABLE 3—X-RAY DIFFRACTION PATTERNS OF SAMPLES OF MORTAR AND CALCIUM SILICATE HYDRATE, AND POSSIBLE ORIGINS OF LINES

Mortars				Calcium silicate hydrate																	
B13V Non-carbonated		B8V Carbon-ated		P76 Non-carbonated		P77 Carbonated		Quartz*		Xonotlite†		Alite†		Platy tober-morite‡		Calcium hydrox-ide§		Vaterite**		Aragon-ite††		Calcite‡‡	
I‖§	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)	I	d, (Å)
		?	10.69																		
		?	8.94																		
vw	8.85																				
w	6.82	w	(7.01	w	6.96	vw	6.86			mw	7.05										
w	6.64	w	(6.66	w	6.70	vw	6.60	harmonic***													
vw	5.80																				
vw	5.60					?	5.49							w	5.5						
w	4.91	vw	4.95	w	4.98											74	4.90				
				vw	4.65																
				vw	4.55																
				?	4.44 †††																
s	4.26	s	4.27	mw	4.27 →	m	4.27	35	4.26	mw	4.27							100	4.28		
vw	4.14	?	4.15																		
vw	4.11			?	4.08																
				vw	3.98	vw	3.98														
w	3.85→	mw	3.87	vw	3.89 →	w	3.86														12 3.86
vw	3.76	?	3.77																		
w	3.64	vw	3.66	w	3.67					ms	3.65										
w	3.58→	mw	3.58		→	vw	3.57											70	3.57		
vw	3.49	vwd	3.50		→	w	3.41													100	3.40
vs	3.35	vs	3.35	vs	3.36	s	3.35	100	3.343												
	→	w	3.30		→	vw	3.28											70	3.27	48	3.29
vw	3.18	?	3.24							ms	3.23										
w	3.12				→	vw	3.15									23	3.11				
w	3.079	vw	3.086	w	3.097					vs	3.07			vs	3.09						
m	3.045→	ms	3.048	mw	3.045 →	ms	3.041					s	3.022					50	3.00		100 3.035
				?	2.969									s	2.99						
				?	2.928																
vw	2.879				→	vw	2.856			m	2.83										
mw	2.786	?	2.791	wd	2.821	vw	2.799					vs	2.776	m	2.82						
mw	2.740	mw	2.740	wd	2.755					mw	2.71	s	2.730					100	2.71		
				?	2.698 →	mwb	2.708													64	2.70
mw	2.631	?	2.654	w	2.637	vw	2.627									100	2.628				

vw	2.612										vs	2.602							
?	2.575			?	2.580	vw	2.587												
w	2.550	vw	2.559	?	2.545					mw	2.51							48	2.49	14	2.495
w	2.500	w	2.500	w	2.505 →	m	2.495														
m	2.460	m	2.462	m	2.466	w	2.464	12	2.458												
vw	2.407	?	2.421	?	2.397	vw	2.425														
vm	2.384	vw	2.384	vw	2.360 →	w	2.377											48	2.36		
		?	2.348		→	vw	2.340														
				vw	2.310																
m	2.283	m	2.287	m	2.289	w	2.289	12	2.282												
mw	2.237	m	2.241	w	2.244	w	2.244	6	2.237							70	2.29			18	2.285
?	2.194	vw	2.220																		
?	2.168	vw	2.191		→	vwbd	2.186				vs	2.185						11	2.19		
m	1.128	m	2.132	w	2.133	w	2.135	9	2.128												
wd	2.092→	mw	2.099	vwd	2.100→	mw	2.104											24	2.10	18	2.095
wd	2.059→	mw	2.069							s	2.04					100	2.06				
wd	2.024	?	2.034																		
		?	2.014																		
m	1.978	m	1.984	w	1.984 →	m	1.984	6	1.980									100	1.98		
vw	1.952	?	1.961							s	1.95										
w	(1.929→	w	(1.929	vw	(1.935→									42	1.927					5	1.927
w	(1.918		(1.914	vw	(1.919	wbd	1.921													17	1.913
w	1.878	w	1.878	wd	1.887 →	mwbd	1.884											64	1.88	17	1.875
?	1.856→	wd	1.855	wd	1.844					mw	1.84		s 1.84			100	1.84				
ms	1.821	ms	1.821	m	1.823	w	1.822	17	1.817									32	1.82		
w	1.804	w	1.808	vw	1.803	vw	1.802							36	1.796						
vw	1.766	vw	1.772									s+	(1.771								
vw	1.745					w	(1.744					s	(1.752					40	1.74		
vw	1.693				→	vw	(1.728			mw	1.710			21	1.687						
mw	(1.673	mw	(1.675	mw	(1.677	w	(1.679	7	1.672												
w	(1.660	w	(1.663	w	(1.663	w	(1.660														

*ASTM X-Ray Powder Data File, Card No. 5-0490. Data of high accuracy.
†Heller, L., and Taylor, H. F. W., *Crystallographic Data for the Calcium Silicates*, Her Majesty's Stationery Office, London, 1956, p. 47 and p. 20.
‡Reference 19.
§Swanson, Howard, E., and Tatge, Eleanor, "Standard X-Ray Diffraction Powder Patterns," *Circular* No. 539, National Bureau of Standards, V. 1, 1953, p. 59. Data of high accuracy.
**ASTM X-Ray Powder Data File, Card No. 2-0261.
††ASTM X-Ray Powder Data File, Card No. 1-0628.
‡‡Swanson, Howard, E., and Fuyat, Ruth K., "Standard X-Ray Diffraction Powder Patterns," *Circular* No. 539, National Bureau of Standards, V. 2, 1953, p. 52. Data of high accuracy.
§§Intensity scale: vs—Very strong; s—strong; ms—medium strong; m—medium; mw—medium weak; w—weak; vw—very weak; vvw—very very weak; ?—presence uncertain; d—diffuse; b—broad.
***Harmonic of very strong quartz line at 3.34A. †††An arrow indicates a definite increase in the intensity of a line through carbonation.

explained by the presence of some xonotlite [$Ca_6(Si_6O_{17})(OH)_2$], tobermorite [$xCaO{\cdot}SiO_2{\cdot}yH_2O$] (probably platy variety, Sanders, and Smothers[19]), and unreacted alite [$Ca_3O(SiO_4)$].

After carbonation (B8V, Table 3) the x-ray pattern of calcium hydroxide is largely removed (note the drop in intensity of lines at about 4.90, 3.10, 2.63, and 1.690Å) and those prominent lines (3.08, 2.79, and 2.61Å) considered to be due to xonotlite, tobermorite, or alite are weakened in intensity. Carbonation has not reduced the intensity of all the calcium hydroxide lines or of all the lines due to the cement minerals because there are many cases where such lines superimpose on lines from quartz and from the carbonation products. After carbonation some lines appear and others increase in intensity; these changes have been indicated by arrows in Table 3 connecting B13V to B8V. They are considered to be due to an increase in the amount of calcite ($CaCO_3$) and the formation of some vaterite (μ-$CaCO_3$) and a little aragonite ($CaCO_3$). It should be noted that the patterns of these forms of calcium carbonate are similar and, if present together, difficult to separate even on x-ray cameras of high dispersion such as the focussing camera used in this study. Interpretation is made even more difficult in the present instance by the strong interfering quartz pattern and the broad and diffuse nature of the lines in the carbonated samples; the latter is probably caused by particles of small size or disordered structures or both. However, the calcite line at 3.86Å and the doublet at 1.93 and 1.91Å, the vaterite lines at 3.57 and 2.06Å, and the aragonite lines at 3.40 and 1.98Å are clearly distinguishable from each other and from the cement minerals. The calcite doublet at 1.93 and 1.91Å is coincident with the line at 1.93Å in calcium hydroxide but the latter is destroyed during carbonation. The aragonite line at 1.98Å superimposes on a weak quartz line but since the quartz is unaffected by carbonation this can be allowed for.

The above lines and the patterns as a whole for all the mortar samples examined and for the gravity fractionated portions of Sample B6 have been carefully considered, together with the results of heat treatment of the mortars, as described later. As a result it has been concluded that for the mortar samples the increase in the carbon dioxide content through carbonation (Table 2) is due to the development of poorly crystallized calcite, vaterite, and aragonite from the decomposition of the cement minerals. That the calcite is poorly crystallized can be judged (Table 3) from the fact that before carbonation the calcite doublet at 1.93 and 1.91Å is clearly resolved; after carbonation it is replaced by a broad and diffuse band. It is difficult to make a quantitative assessment of the amount of each phase present, but aragonite is subordinate to calcite and vaterite. The conclusion that the aragonite content is low is based on the absence of a well-

defined line at 3.40Å and no change in the intensity of the line at 1.98Å through carbonation. The conclusion that vaterite is present in some quantity is based on the increase in the intensity of the lines at 3.58 and 2.06Å through carbonation. It should be noted that the x-ray patterns of samples exposed to atmosphere for 3 and 30 months and of those exposed artificially to carbon dioxide were essentially the same.

The x-ray powder pattern of the noncarbonated sample of calcium silicate hydrate is similar to that of the mortar samples just discussed (Table 3). The sample contains quartz, a trace of calcium hydroxide, and some calcite. Fibrous tobermorite is probably present in addition to xonotlite and a trace of platy tobermorite and/or hillebrandite ($2CaO \cdot SiO_2 \cdot H_2O$). Here also it is difficult to be certain of the particular compound formed since basal spacings characteristic of cement minerals are seldom present in the x-ray powder patterns of samples autoclaved for short periods. As in the case of the mortars, the calcium silicate hydrate minerals are largely destroyed in carbonation (note in Table 3 that lines at 3.67, 3.10, and 2.76Å are present in Sample P76 but absent in Sample P77) and are replaced by other minerals (as judged by the increase in intensity of some lines and the appearance of others as indicated in Table 3). The replacement minerals are considered to be calcite, which increases in amount through carbonation (based on the lines at 3.86 and 1.92Å in Sample P77), much aragonite (based on the lines at 3.41, 2.38, 2.19, 1.98, and 1.73Å in Sample P77), and a little vaterite (based on the very weak line at 3.57Å in Sample P77, the 2.06Å line being too weak to appear). The x-ray diffraction lines of the products of carbonation are, like those of the mortars, broad and diffuse.

It would appear from the x-ray results that the main distinction that can be drawn between the carbonated mortar samples and the sample of calcium silicate hydrate is that in the former, vaterite is dominant over aragonite, whereas in the sample of calcium silicate hydrate, aragonite is dominant over vaterite.

In analyzing the x-ray results the possible presence of silicate minerals containing carbon dioxide such as scawtite [$6CaO \cdot 4SiO_2 \cdot 3CO_2$], spurrite [$2Ca_2SiO_4 \cdot CaCO_3$], and tilleyite [$Ca_5(Si_2O_7)(CO_3)_2$] have been considered. A comparison of the x-ray data on scawtite,[20,21] with the present results indicates that this mineral could not be present in any quantity in the carbonated samples. In the case of spurrite the two main lines at 3.03 and 2.70Å (ASTM card index 4-0640) are coincident with lines from calcite, aragonite, and vaterite, the presence of which is certain, so that it is not likely that this mineral is present in any quantity. Although there are no x-ray powder data available for tilleyite, its presence is also unlikely in any quantity, since the origins of most of the x-ray diffraction lines are satisfactorily accounted for.

However, the x-ray results do not exclude the possibility that poorly crystallized or amorphous silicate minerals containing carbon dioxide could be present. There is no evidence from heated samples (to be described below) to suggest that such minerals are present. The chemical analyses in Table 1 indicate that the mortar samples could contain a few percent of alkali carbonates. The quantities involved, however, would be too small to be readily detected by x-ray methods in such complex mixtures.

Heat-treated samples — Since the x-ray results indicate that the carbon dioxide taken up by samples of mortar and of calcium silicate hydrate is largely in the form of the calcium carbonate minerals vaterite, aragonite, and calcite (all in finely divided form or disordered) it should be possible to confirm by x-ray analysis of heated specimens that all three minerals are removed from samples by heating at temperatures below the dissociation temperature of well-crystallized calcite. This has been investigated in two ways.

Firstly, samples of both mortar and calcium silicate hydrate were x-rayed after the carbon dioxide below 500 C had been determined in obtaining the data for Table 1. The x-ray results showed that the vaterite and aragonite had disappeared (owing to conversion to calcite) and the amount of calcite had been considerably reduced, much of that remaining being of a well-crystallized form.

Secondly, for the light fraction of mortar Sample B6 and the carbonated sample of calcium silicate hydrate (P77), material for x-ray examination was removed from the sample block during differential thermal analysis at the temperatures marked on Curves C and K, Fig. 1. The temperatures chosen were on either side of endothermic or exothermic reactions so that an examination could be made of the processes taking place during the reactions. The results are illustrated diagrammatically in Table 4. They show that, in samples of both mortar and calcium silicate hydrate, poorly crystallized calcite, including that developed from vaterite and aragonite, reacts with the siliceous residue from which it was derived and with which it is still intimately associated. This reaction yields larnite (β-$2CaO{\cdot}SiO_2$) at a temperature well below that at which well-crystallized calcite dissociates. It is to be noted that larnite is only meta-stable at room temperature and exists there only through the presence of stabilizers.[22,23] In the present instance the samples were quenched rapidly from about 700 C; when they were allowed to cool slowly the larnite did not appear. Neither did larnite appear in the noncarbonated sample of calcium silicate hydrate (P76) when a portion was taken from the differential thermal equipment at a temperature of 650 C (J, Fig. 1). The formation of larnite, unlike that of wollastonite ($CaO{\cdot}SiO_2$), is not exothermic but follows the endothermic reaction due to the dissociation of the poorly crystallized

TABLE 4—PHASE CHANGES UNDERGONE BY SAMPLES OF CARBONATED MORTAR AND CALCIUM SILICATE HYDRATE (CSH) DURING DIFFERENTIAL THERMAL ANALYSIS

Material	Temperature range, deg C	Type of reaction	Phases					
			Quartz	Calcite well crystallized	Siliceous residue	Calcite poorly crystallized	Aragonite	Vaterite
CSH Mortar	20-500 20-600		Unaffected				Partly transformed	
CSH Mortar	500-710 600-750	Endothermic	Unaffected			CO_2 loss Larnite (β-$2CaO \cdot SiO_2$)	Completely transformed	
CSH Mortar	710-800 750-850	Endothermic	Unaffected	CO_2 loss CaO		Unaffected		
CSH	800-900	Exothermic	Quartz Cristobalite?	Wollastonite ($CaO \cdot SiO_2$) Pseudo-wollastonite ($CaO \cdot SiO_2$)				
Mortar	850-900	Exothermic	Quartz	Wollastonite ($CaO \cdot SiO_2$) Pseudo-wollastonite ($CaO \cdot SiO_2$) Gehlenite ($2CaO \cdot Al_2O_3 \cdot SiO_2$)		Larnite (β-$2CaO \cdot SiO_2$) Recrystallized		

calcite. In the final exothermic reaction for samples of both mortar and calcium silicate hydrate, wollastonite, and pseudo-wollastonite ($CaO \cdot SiO_2$) are formed from a reaction between larnite and the remainder of the siliceous residue. They are formed also by some quartz reacting with CaO freed endothermically from well-crystallized calcite. Since the mortar samples contain an appreciable amount of alumina and iron oxide, probably present after carbonation[11] as aluminum hydroxide and hydrated ferric oxide, respectively, the final exothermic reaction results in a more complex mixture of phases than for the calcium silicate hydrate. Gehlenite ($2CaO \cdot Al_2O_3 \cdot SiO_2$) appears in addition to wollastonite and pseudo-wollastonite, and larnite persists in a recrystallized form.

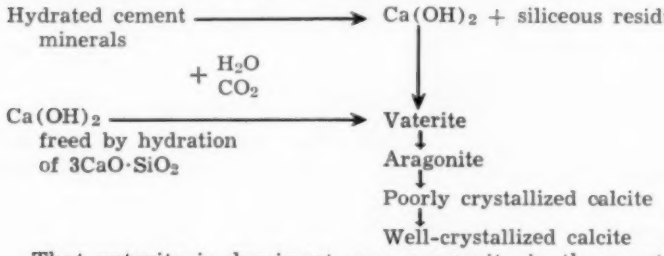
DISCUSSION

Results in the present work have indicated that the greater part of the carbon dioxide taken up by samples of mortar and of calcium silicate hydrate exposed to the atmosphere or to a stream of carbon dioxide is not held as well-crystallized calcite but as poorly crystallized vaterite, aragonite, and calcite and that the latter decomposes at a low temperature when the sample is heated. A somewhat similar conclusion has already been reached by Gaze and Robertson[13] who described the carbonate material as a "two-dimensional calcite" which requires less energy to break it down to CaO and CO_2 than three-dimensional calcite. However, the x-ray photographs taken in the present work do not indicate two-dimensional lattices but very finely divided vaterite, aragonite, and calcite with three-dimensional lattices. The calcite decomposes at a lower temperature than more crystalline varieties because of the particle size effect,[24] and because it has grown on a siliceous residue (resulting from calcium silicates decomposed by hydration and carbonation) with which it readily reacts on rapid heating to form larnite (β-$2CaO \cdot SiO_2$). Calcite and quartz have similarly been noted to react to form wollastonite ($CaO \cdot SiO_2$) at temperatures well below the decomposition temperature of calcite alone.[25,26] Soluble salts present could also influence the thermal effect accompanying the decomposition of the calcite[27] and provide a possible stabilizer for β-$2CaO \cdot SiO_2$. On the question of the anomalous dissociation of carbonate minerals, Feilchenfeld and Eden[28] have described an intimate association of carbonate and apatite which has peculiar thermal effects.

The calcium carbonate minerals have formed in the mortars both from $Ca(OH)_2$ and from the decomposition of hydrated cement minerals (largely hydrated calcium silicates in portland cement mortars). The $Ca(OH)_2$ is formed during hydration of the cement by conversion of $3CaO \cdot SiO_2$ (alite) to $2CaO \cdot SiO_2 \cdot xH_2O$ [29] and the amount present in the mortars studied as well as the amount of calcium carbonate that

could be formed from it are set out in Table 2. There is clearly insufficient $Ca(OH)_2$ to account for all the CO_2 taken up by the mortar in the form of $CaCO_3$ so that there must also be carbonation of the hydrated calcium silicate minerals to meet the CaO deficiency. This has already been noted.[3,8,11-13] In the sample of calcium silicate hydrate the calcium carbonate minerals all formed during carbonation at the expense of the calcium silicate hydrate mineral.

It has been observed[9,10,12] that carbonation of portland cement mortars requires the presence of a certain amount of moisture before reaction will take place. This suggests that the carbon dioxide is dissolved in the pore water and that chemical attack of the $Ca(OH)_2$ and cement minerals by carbonic acid then takes place. This would offer an explanation for the association of poorly crystallized vaterite, aragonite, and calcite found in samples of carbonated mortar and of calcium silicate hydrate. The conditions of the formation of vaterite and aragonite by precipitation from solution[30] suggest that vaterite can form first, then be transformed into aragonite which is then transformed into poorly crystallized calcite, thus obeying Ostwald's law of successive reactions. Unpublished results of the authors show that if calcined calcite is allowed to recarbonate in air the CaO first hydrates to form $Ca(OH)_2$ which is then converted to carbonate by forming aragonite and some vaterite. These subsequently transform into poorly crystallized calcite. On the other hand, when calcined calcite is allowed to recarbonate in dry CO_2,[31] well-crystallized calcite is formed. It can thus be argued that for cement minerals carbonating in the presence of moisture the following processes could take place:



That vaterite is dominant over aragonite in the mortar samples and aragonite dominant over vaterite in the sample of calcium silicate hydrate could mean that these reactions have proceeded to different stages at the time of examination. Alternatively, it could indicate that aragonite need not necessarily have been formed from vaterite, since the type of calcium carbonate that precipitates is very dependent upon pH conditions.[30] A high pH favors the formation of vaterite. The mortars would presumably develop a higher pH value than would the sample of calcium silicate hydrate when moistened, since, before carbonation, they contain free calcium hydroxide. However, the presence

of much calcium hydroxide and aragonite in the calcined calcite carbonated in air is not consistent with this argument and indicates that other factors are also effective. That mortar samples carbonated in air for 3 and 30 months differed only in the amount of well-crystallized calcite suggests that the amounts of vaterite, aragonite, and poorly crystallized calcite formed are not related to each other by a step reaction, though all can be converted to well-crystallized calcite with time. This aspect of the problem could be studied further.

Since vaterite is transformed into calcite at a lower temperature than that at which aragonite is,[30,32] it seems unlikely that the difference in the forms of calcium carbonate present in the samples of mortar and of calcium silicate hydrate will explain why the latter loses carbon dioxide and forms larnite at a lower temperature than the mortar samples. It seems more likely that in the mortars the presence of phases other than calcium silicate hydrate minerals results in the carbon dioxide being lost at a higher temperature.

In studying the loss of carbon dioxide from heated carbonated mortars Kroone and Blakey[12] made a distinction between carbon dioxide lost below and above 500 C. They chose this temperature as that at which well-crystallized calcite has a negligible dissociation pressure (0.1 mm[33]) and should not decompose. However, this argument ignores the possibility of well-crystallized calcite reacting with quartz[25,26] at a temperature lower than the dissociation temperature of pure calcite, and does not allow for poorly crystallized calcite losing carbon dioxide both below and *above* 500 C. Consequently, the distinction made by Kroone and Blakey does not differentiate between poorly and well-crystallized calcite, and should be mainly used simply as a means of comparing different samples.

CONCLUSIONS

The carbon dioxide taken up by carbonated samples of mortar and of calcium silicate hydrate exposed to the atmosphere or to a stream of carbon dioxide is present as calcium carbonate in the form of poorly crystallized vaterite, aragonite, and calcite. Small amounts of well-crystallized calcite are present, but there is no great increase in this due to carbonation.

There is no evidence to suggest than any carbon dioxide is held by the crystalline silicate minerals scawtite, spurrite, and tilleyite, nor is there any evidence to indicate that it is held by amorphous silicates of similar composition.

In carbonated mortars, vaterite is dominant over aragonite, but in the sample of calcium silicate hydrate aragonite is dominant over vaterite. This difference may result from different conditions of pH during carbonation. On the other hand, it may merely represent different

stages in the progress of the carbonation reaction at the time of examination.

The calcium carbonate minerals have formed both from calcium hydroxide, freed during the hydration of alite ($3CaO \cdot SiO_2$), and from the decomposition of hydrated cement minerals.

Since carbonation will take place only in the presence of moisture it is suggested that the mechanism is an action of carbonic acid on cement minerals resulting in the following probable sequence of events: Cement minerals → Siliceous residue + Calcium hydroxide → Vaterite + Aragonite + Poorly crystallized calcite → Well-crystallized calcite.

Heating carbonated samples of mortar and of calcium silicate hydrate transforms poorly crystallized vaterite and aragonite to poorly crystallized calcite. If the heating is carried out rapidly the calcite reacts (at a temperature well below the dissociation temperature of well-crystallized calcite) with the siliceous residue with which it is intimately associated to give the unstable larnite (β-$2CaO \cdot SiO_2$) which, at room temperature, is discernible in quenched samples. At higher temperatures the larnite in the mortar samples recrystallizes, but in the sample of calcium silicate hydrate it is converted to wollastonite ($CaO \cdot SiO_2$).

The samples of mortar decompose at a higher temperature than does the sample of calcium silicate hydrate. This is probably due to the polyphase nature of the mortars which results in a more stable system.

Sufficient distinction between poorly crystallized and well-crystallized calcite cannot be made simply by the determination of carbon dioxide present below and above 500 C in carbonated samples of portland cement mortar and of calcium silicate hydrate.

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Anchor Bolts in Massive Foundations

By E. SCHECHTER*

C. A. LEE's "Design of Anchor Bolts in Foundations"† is a challenging approach to a difficult problem. It also suggests an answer to the problem of rock anchors for major structure foundations.

Consider the problems now being faced as the new techniques of prestressing are used to anchor dams and dam spillways. An example of current practice in bonded anchorage for dams is shown in Fig. 1. High-strength wires or strands, formed into a shape designed to provide extra shear and bond values, are placed in a cored hole. A grout plug is poured, and when the grout has gained strength the unit is post-tensioned.

An alternate to this scheme makes use of the basic thinking described by Lee for foundation bolts. In this instance a bar post-tensioning unit having an anchor plate attached to the bottom end is treated with a bond breaking compound within the grout plug as in Fig. 2. When the concrete in the plug has reached required strength, post-tensioning of the bar tendons is transmitted directly to the bottom of the plug.

The prestressing force in these two examples is transmitted to the anchoring rock in radically different ways.

In Fig. 1 the force is transmitted from the top through tension in the tendons and transferred to the concrete plug through bond between the tendon and concrete. The concrete in the plug has been placed when the tension in the tendon is zero. As tension is applied to the tendon its diameter will be slightly reduced. This means that surface bond is destroyed for some distance below the top of plug. If wire strands are used to form tendons, then any stress transfer to the plug is by mechanical means only. In a further distance down, surface bond strength will prevail and full transfer will have been accomplished. It is questionable if this distance can be accurately determined. If this distance is less than the length of plug required to develop sufficient shear

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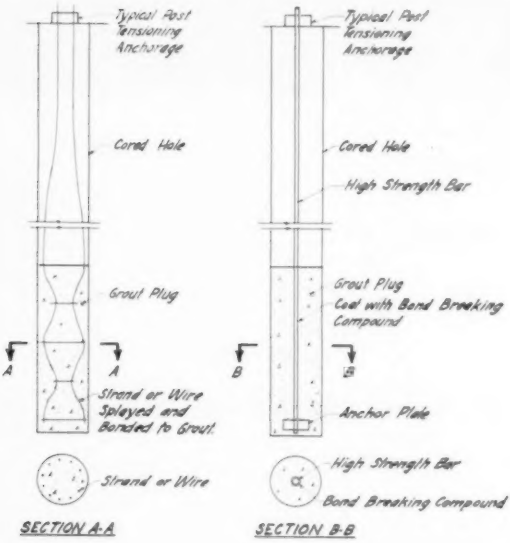

Fig. 1 (left)—Bonded prestressed anchorage for dams using wires or strands. Fig. 2 (right)—Prestressed anchorage for dams or similar structures using bar post-tensioning unit

between plug and cored hole, it is reasonable to assume that tension must develop in the plug. Recognizing this, engineers must assume a depth of plug greater than that which should be required for safe anchorage of the plug.

In Fig. 2, the prestressing force is also transmitted through tension in the tendons, but this force is anchored at the bottom of the plug and transferred to the plug by compression. The plug, acting in compression, transmits stress through shear to the cored hole. In this instance the only fully indeterminate factor is the bond value between the plug and the cored hole, which is also true for Fig. 1. However, no additional length of plug must be provided to account for tendon bond transfer as in Fig. 1.

In the anchor described by Fig. 2, high bearing stresses between steel plate and grout develop at the bottom of the plug. As the grout is fully restrained and no movement can be transmitted to the grout, this high bearing stress must be dissipated and distributed triaxially through the plug and into the rock.

The principle disadvantages of the conventional method described in Fig. 1 are that high tensile stresses occur in the concrete at the top portion of the plug, bond length is indeterminate, and deeper than necessary anchors are used. The principle disadvantage of the method described in Fig. 2 is that high bearing stress occur at the bottom of the plug. As this latter is solely a compressive stress fully contained at the bottom of a plug it is this writer's opinion that the scheme described in Fig. 2 will provide a more predictable and superior anchorage.

The Problems and Practices section (which appears in Concrete Briefs section as space allows) is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Perhaps YOUR solution is a better way of handling some everyday problem.

Random-Width Lumber and Cardboard Cartons Cut Formwork Costs

TWO COST-SAVING building techniques employed in the construction of a five-story reinforced concrete Air Force tower were (1) random-width, rough-cut lumber for wall formwork, and (2) cardboard boxes with egg-crate cores used to form "waffle" floors.

The installation, at Thomasville, Ala., will serve as a prototype for other sites. Over-all savings developed on the initial design may therefore be multiplied with each installation.

Random-width forms

The rough-cut lumber of unequal width was used to form deliberate steps in the wall (Fig. 1). In this way, any unevenness which would normally be visible goes unnoticed, and the high cost of hand rubbing and finishing was eliminated. Also, by rough-cutting the lumber, the swirls and grain structure of the wood were picked up by the concrete and created a decorative effect.

Forms for exterior walls between columns, spaced at about 20 ft, were made of No. 2 grade or better lumber, 6 to 12 in. wide, and between ¾ and 1½ in. thick, with horizontal joints. The form panels were designed and spaced to achieve a minimum wall thickness of 10 in. and were made up of varying thicknesses of lumber held together by 1 x 3-in. battens. The panels were built up into 2 ft high sections as shown in Fig. 2. Plywood strips nailed along the joints prevented seepage of the concrete.

Panels were coated with form oil, hoisted into position, and secured with form ties and wales. Wales were left loose until studs had been inserted between them and the composite panel; form ties were then tightened.

This procedure, while slightly different from usual construction techniques, had a distinct advantage. Normally, panels would be nailed to studs and erected as a unit. However, by sliding studs into place and confining the size of the panel, the crane required to handle heavy forms could be eliminated. Two simple slings raised panels to the required elevation (Fig. 3).

Forms were left up for 7 days. After forms had been removed, cone holes were filled. They were not rubbed smooth, but were "board-marked" to blend with the surrounding rough-textured finish of the wall.

Cardboard forms

The other technique which cut costs and provided flexibility on the project was the use of cardboard boxes with egg-crate cores to form waffle-like floors. The floors were designed with

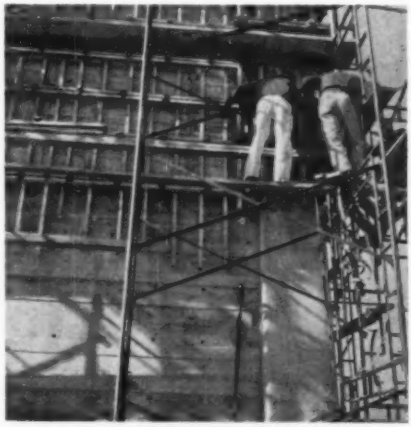

Fig. 1—Finished appearance of "stepped" wall is visible where forms have been removed. Tie cone holes are filled later and "board-marked" to blend with wall texture

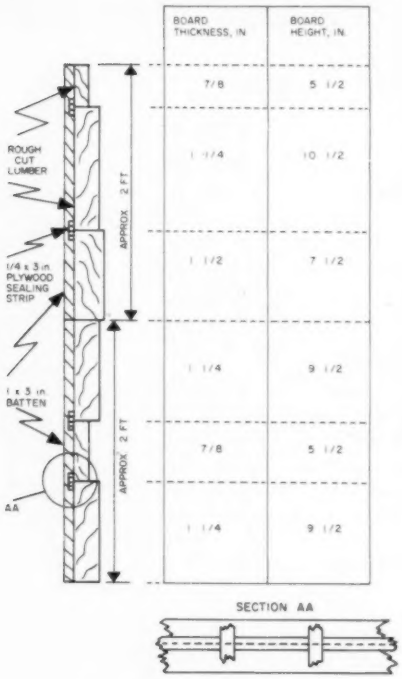

Fig. 2—Form panel layout

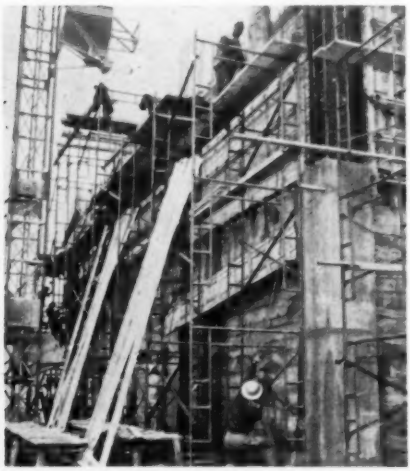

Fig. 3—Built in 2-ft sections, the form panels could be raised into position by two men

concrete ribs running in two directions and were reinforced with wire mesh in the top slab. This design provided for the rigidity required by the electronic equipment, and was economical. In addition, the thin slab areas between waffle ribs may be omitted to leave holes of various sizes without seriously affecting the integrity of the structure.

The concrete ribs and floor slab were concreted integrally with the walls of the floor below and were reinforced with #4 to #8 bars. The ribs thus formed a checkerboard design, 30 in. on center. Spacing and depth were achieved by means of the cardboard forms which were simpler and faster to fabricate than the special wood or metal ones which would otherwise have been required. Cardboard shells were taped with 3-in. waterproof, pressure-sensitive tape. A honeycomb inner core was then assembled from die-cut sections and inserted in the shell. It took only 3 min to make up one complete unit; cost of a 24 x 24 x 14-in. core was 25 cents.

Container shells were already wax-impregnated when received. However, to guard them further against possible loss of strength from inclement weather, bottoms were immersed in hot parafffin at the job site. Experience showed that even after a heavy rain workers could walk over the boxes without causing damage. The cores supported a concrete floor slab with a maximum thickness of 3 in.

Besides the advantages of simplicity, cost, and time, the cores were also stripped easily. The egg-crate core was removed with a pair of pliers and was then ready for reuse.

Credits

Burns and Roe, Inc., New York consulting engineers, designed the structure for the Surface Armament Division, Sperry Gyroscope Co., system manager of the program for this air-search radar installation.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove controversial.

All ACI members are invited to participate — either by submitting an inquiry, or even better, by telling JOURNAL *readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

Q. Can you give some information concerning the calculation of pressures in tall silos?

A. Some recent information on this problem can be found in "Concrete Silos," by T. J. S. Mallagh, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 84, 1958, p. 123. The article discusses fairly thoroughly pressures in deep silos although it does not give any fixed values for various materials.

The following discussion has been excerpted from that paper with the permission of the author and the Institution of Civil Engineers of Ireland.

If the depth is at least two to three times the greatest lateral dimension, silos can be classified as deep for the purpose of calculating the pressure of the contained material. For such silos, the horizontal pressures to be resisted by the walls are generally determined by either Janssen's or Airy's formula. Both these formulas have a theoretical oasis, the former being deducted from the Rankine expression of $K = (1-\sin\phi)/(1+\sin\phi)$, for active pressure, and the latter from Coulomb's wedge theory. Both formulas take into account friction between the contained material and the walls, and therefore can be used to determine vertical loading transferred to the walls and weight carried by the bottom of the bin directly.

The Reimberts* have deduced, to cover for static conditions in a bin, the following formula:

$$P_z = P_{max}\left[1-\left(\frac{Z}{A}+1\right)^{-a}\right]$$

where P_z = horizontal pressure at depth z

$$P_{max} = \frac{d\,D}{4\tan\phi_1}$$ = value of P_z at infinite depth

d = density of contained material

D = diameter of bin

ϕ_1 = angle of wall friction

Z = depth from base of cone at top of material

$$A = \frac{D}{4\tan\phi_1\tan^2\left(\frac{\pi}{4}-\frac{\phi}{2}\right)} - \frac{h}{3}$$

ϕ = natural angle of repose of material

h = height of cone at top of material

For maximum depth, this formula gives the same result as the Janssen, and there is close agreement between the results for lesser depths.

The results given by any formulas will depend on the values taken for the constants. These can, to some extent, be determined by experiments, but some rather inconsistent results have been thus obtained. In particular, measurements of vertical and horizontal pressures have not given values for K,

*Reimbert, M. and A., *Silos, Traité Théorique et Pratique*, Editions Eyrolles, Paris.

in the expression $K = (1 - \sin \phi) / (1 + \sin \phi)$, in accordance with the values of ϕ, whether the latter is deduced from angles of repose or otherwise.

Nevertheless, many measurements of horizontal and vertical pressures, in both models and actual silos, have given results sufficiently close for practical purposes to those deduced by the classical formulas (see Ketchum*).

From time to time it has been suggested that the usual formulas only apply to static conditions, and that on emptying or filling a bin, horizontal pressures, in particular, can be over 100 percent above those obtained from the usual formulas. Ketchum's conclusions, after carrying out scale model investigations and analyzing results obtained by others, were that, provided the discharge opening was central and draw-off was no greater than the highest practical rate, any increases over the pressures for static conditions (these being in accordance with the usual formula) would be a maximum of 10 percent.

The Reimberts in their recently published book have resurrected this controversial question of greatly increased pressures in silos during discharge. They have deduced by means of electric strain gages, incorporated in the walls of actual silos, pressures during discharge of more than twice those due to the contained material at rest, the static pressures being in accordance with those given by their formula.

A comparison has been made by the author of the pressure in a 20-ft diameter bin containing wheat according to the Janssen, Airy, and Reimbert formulas, with the same equivalent constants for internal and wall friction being used in each case. It was found that, with the factor to cover for dynamic effect, the Reimberts' formula gave the greatest pressure relatively high in the silo. For the upper portion of a silo, the Reimberts' figures covering for dynamic effect were appreciably in excess of what any of the usual formulas for pressure in a bunker or shallow silos would indicate. Taking this and the results obtained by others into account, it is difficult to justify adoption of the pressure they advocate.

To cover the possibility of the dynamic effect, the author would suggest designing on an ultimate load factor, determining appropriate factors of safety by "weighting" on the lines suggested by Baker.† The pressures deduced by the Reimberts to cover dynamic effect could be used if the discharge was not to be central or if there were other conditions likely to give rise to a dynamic effect. Otherwise pressures given by the usual formula could be used, the "load condition" being appropriately weighted. The use of the ultimate load factor for design would only have significant effect if the silo walls were subjected to primary bending. For a circular prestressed silo, load factor design is the obvious approach (with relevance to internal pressure), but the main point the author wishes to make is the use of a factor determined by weighting in relation to the uncertainty of the correct value to take for internal pressure.

There is little evidence that noncentral filling gives rise to excessive pressures in the same way that noncentral discharge does. Uneven filling can, however, cause unsymmetrical loading of the silo walls and in a large silo the secondary stresses caused by this may be quite significant.

* * * *

Q. "Building Code Requirements for Reinforced Concrete (ACI 318-56)," Section 701, permits the use of certain coefficients provided spans are approximately equal and the uniformly dis-

*Ketchum, M. S., *Design of Walls, Bins and Grain Elevators*, McGraw-Hill Book Co., New York.

†Baker, A. L. L., *The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames*, Concrete Publications, Ltd., London.

tributed live load does not exceed three times the dead load.

A moment distribution for the above case indicates that if a live load equal to dead load is placed on alternate spans starting with the exterior span, negative moment will exist in the first interior unloaded span. Therefore a live load three times the dead load would create a large negative moment in this span. The moment coefficients given in Section 701 make no provision for this negative moment.

I would like to have any information available as to how the three to one ratio of live to dead load was obtained and the reason why this negative moment can be neglected.

A. Quite a discussion could be written about the determination of moments in continuous beams and particularly in monolithic frames. The ACI Code, Section 701(a), recommends a frame analysis by the elastic theory as a general approach to any problem. Then, in Sections 701(b) and 701(c), it suggests certain rather approximate arbitrary coefficients which experience has demonstrated will be reasonably satisfactory under certain limited conditions that are fairly clearly set forth.

These arbitrary coefficients are admittedly approximates, but years of use on fairly regular building frames with working stress designs following the requirements of Chapter 7 have produced satisfactory, economical buildings. Mathematically, any given set of coefficients can only apply to a special condition of loads, spans, and restraints.

It can be shown that under the most adverse conditions of maxima and minima, greater moments could theoretically be obtained, and the Code recommends that the designer makes his own elastic analysis. However, unless a designer is particularly experienced, the cost of preparing such frame analyses is almost prohibitive, and, as mentioned, the arbitrary coefficients within their limitations will do a fairly satisfactory job. They are definitely empirical — the result of years of practical experience by a large committee.

However, a few remarks are pertinent relative to the so-called exact method of elastic frame analysis. The supposedly accurate method has many complications, such as:

1. *Width of support*—The continuous beam is usually analyzed with a hinge at each support and is then built monolithic into a fairly wide pier, wall, or column. Some correction, such as the deduction of $Va/3$ is necessary to reduce from the peak moment to a design moment.

2. *Column stiffness* — Continuous beam analysis is often made with no regard to the restraint of the supporting columns. Such a procedure is probably safe and satisfactory, but not of such high degree of precision as the computer often imagines.

3. *End restraint* — Generally computations are based on the continuous beam free on its outer end. Then it is cast monolithic with a column or embedded in a wall. A considerable restraint is developed, which again somewhat vitiates the high degree of precision the computer has visualized.

4. *Modulus of elasticity* — A somewhat more subtle disturbing element is the fact that the modulus of elasticity of the concrete can vary along the length of the structure with age, humidity, and other ambient conditions, so that assuming a constant E_c is not as precise as one might wish.

5. *Moment of inertia* — Often frame analyses are made with constant I throughout the length of the member. A reinforced concrete beam, which is usually a T-beam and often has steel cut off or bent up within the span, has varying moments of inertia, and assuming a constant I is, at best, an approximation, while attempting to deal with the variable moment of inertia simply adds computational problems that hardly seem justified.

6. *Transformed area* — Considerable differences of opinion exist as to whether the most accurate prediction of deflection results from a consideration of the gross concrete area independent of reinforcement, the transformed area, or an intermediate approximation, and this is tied up with whether the section has cracked or is still intact.

7. *Creep* — An added difficulty is the matter of creep, where the time yield of concrete may double or triple the short-time deflection. This is, to some extent, dependent on the internal stress for the time duration and consequently will cause varying modifications throughout the length.

8. *Joints* — Since construction joints are almost inevitable in most frames, some thought has to be given to whether the angular rotation at a joint is any greater than in the material on either side.

9. *Loads* — The theoretical maximum checkerboard location of load patterns is easy to visualize, but there is a question of probabilities and as to how frequently this situation may actually be realized in the field.

10. *Connections* — Most frame analyses, when they do include the effect of columns, are based on the assumption that the beam-column connection is infinitely stiff, that there is no play in the angular connection. This is probably a reasonable assumption, but mathematically it is involved in the exact analysis (unless, as in some structural steel designs, you place an ultimate moment value on the connection).

11. *Discrepancy* — As has been pointed out elsewhere, reinforced concrete is not a highly precise material. I think it justifies about "two-figure precision." The various batches of concrete, the methods of construction, the building of forms, placing of steel, and curing all involve minor opportunities for variation, and while these are in no way detrimental to reinforced concrete construction, the raising of bars even ¼ in. in a 4-in. slab will affect the moment value about 8 percent, and hence the relative stiffness about 20 percent.

12. *Settlement* — Unequal settlement of the supports can affect the distribution of moments materially. Of course, the greater the settlement, the greater the imbalance.

All of the above comments are thrown out merely to suggest that the high degree of precision that one attributes to a precise elastic frame analysis is probably nonexistent. This does not mean that we and most other designers do not make such analyses — because we do — but the results are viewed in the light of the above and other factors.

More and more, I think reinforced concrete design is an art rather than a science. One must cultivate a feel for the behavior of concrete and use mathematical approaches only as a guide to his knowledge and experience.

After all of this, I can only return to the first statement, that buildings designed within the limitations of Sections 701(b) and 701 (c), if carried out according to all of the requirements of the Building Code, will give satisfactory performance; there is nothing to prevent, but everything to urge, a designer to make his own frame analysis and to provide reinforcing steel at every point where he feels that it is desirable.

One last thought: generally a structural steel member is symmetrical in cross section and can take positive or negative moments equally well, but a reinforced concrete member is tailor-made for the job and will only take moments where the designer has provided the necessary reinforcement.

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Chairman, ACI Committee 318
Consulting Engineer
Toledo, Ohio

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Periodicals Regularly Scanned For Reviews

The many requests for information concerning the source material of reviews appearing in the Current Reviews section indicate the desirability of publishing a list of those periodicals, bulletins, and proceedings being *scanned regularly* for articles dealing with the field of concrete. If all miscellaneous magazines, bulletins, reports, and books were included (which are reviewed only as they come to the attention of the voluntary reviewers or editors) the following list would cover well over 200 periodicals and/or publishing, educational, and research organizations. The following list includes publishers' addresses for convenience should correspondence with them be desired, since copies of articles or books reviewed in the JOURNAL are *not available* through ACI. In most cases they can be obtained directly from the original publishers.

American Ceramic Society Bulletin — American Ceramic Society, 4055 North High St., Columbus 14, Ohio

American Society of Civil Engineers, Proceedings — American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.

ASTM Bulletin—American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

American Society for Testing Materials, Proceedings — American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

Annales, Institut Technique du Bâtiment et des Travaux Publics—L'Institut Technique du Bâtiment et des Travaux Publics, 6 Rue Paul Valéry, Paris 16e, France

Annales des Ponts et Chaussées—Commission des Annales a l'Ecole Nationale des Ponts et Chaussées, 28 Rue des Saints-Peres, Paris 7e, France

Applied Mechanics Reviews—American Society of Mechanical Engineers, 29 West 39th St., New York 18, N. Y.

Architectural Forum — 9 Rockefeller Plaza, New York 20, N. Y.

Architectural Record — 119 West 40th St., New York 18, N. Y.

Archiwum Inzynierii Ladowej — Kultury i Nauki, Pokoj 2319, Warsaw, Poland

Australian Journal of Applied Sciences —Commonwealth Scientific and Industrial Research Organization, 314 Albert St., E. Melbourne, C 2, Victoria, Australia

Battelle Technical Review — Battelle Memorial Institute, 505 King Ave., Columbus 1, Ohio

Bauingenieur, Der—Heidelberger Platz 3, Berlin, Wilmersdorf, Germany

Bauplanung-Bautechnik — Unter den Linden 12, Berlin W8, Germany

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A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 12, June 1960, *Proceedings* V. 56. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Bauzeitung, Die — Hospitalstrasse 12, Stuttgart N, Germany

Béton Armé—22 Rue de L'Arcade, Paris 8e, France

Beton Herstellung und Verwendung—Beton-Verlag GmbH., Kaiser- Friedrick-Ring 14, Dusseldorf Oberkassel, Germany

Betongen Idag—Norsk Cementforening, Gaustadalleen 30, Oslo - Blindern, Norway

Beton i Zhelezobeton—Moscow, Russia

Betonstein-Zeitung — Kleine Wilhelmstrasse 7, Wiesbaden, Germany

Beton-Teknik — Christians Brygge 28, Copenhagen V, Denmark

Beton- und Stahlbetonbau—Hohenzollerndamm 169, Berlin - Wilmersdorf, Germany

British Constructional Engineer, The—32 Southwark Bridge Road, London, SE 1, England

Building Construction Illustrated—5 South Wabash Ave., Chicago 3, Ill.

BRI Abstracts of Building Science Publications — Building Research Institute, 2101 Constitution Ave., Washington 25, D. C.

Building Research Station Digest—Building Research Station, Garston, Watford, Herts, England

Builleten Stroitelnoi Tekhniki — Moscow, Russia

Bulletin de la Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions — Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions (RILEM), 12 Rue Brancion, Paris 15e, France

Cahiers du Centre Scientifique et Technique du Batiment—Centre Scientifique et Technique du Batiment, 4 Avenue du Recteur-Poincaré, Paris 16e, France

Cement—Herengracht 507, Amsterdam C, Netherlands

Cement and Lime Manufacture—14 Dartmouth St., London, SW 1, England

Cement och Betong — Kolga-huset, Postgiro 1808, Malmo, Sweden

Cemento, Il—Viale Lunigiana, 16 Milano 510, Italy

Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, Bulletin — Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, 197 Boulevard Saint-Germain, Paris 7e, France

Ceramic Abstracts—(see *Journal of the American Ceramic Society*)

Civil Engineering — American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.

Civil Engineering and Public Works Review—8 Buckingham St., London, WC 2, England

Commonwealth Engineer, The — 349 Collins St., Melbourne C 1, Australia

Concrete—400 W. Madison St., Chicago 6, Ill.

Concrete and Constructional Engineering—14 Dartmouth St., London, SW 1, England

Concrete Construction Magazine—P. O. Box 444, Elmhurst, Ill.

Concrete Products—79 W. Monroe St., Chicago 3, Ill.

Construction—92 Rue Bonaparte, Paris 6e, France

Construction Equipment—205 East 42nd St., New York 17, N. Y.

Constructional Review—14 Spring St., Sydney, N.S.W., Australia

Consulting Engineer — 227 Wayne St., St. Joseph, Mich.

Contractors and Engineers—470 Fourth Ave., New York 16, N. Y.

Crushed Stone Journal, The—National Crushed Stone Association, 1415 Elliot Place, NW, Washington 7, D. C.

Danish National Institute of Building Research, Building Research Reports —Danish National Institute of Build-

ing Research, 20 Borgergade, Copenhagen K, Denmark

Deutsche Bauzeitschrift—Eickhofstrasse 14/16, Gütersloh, Germany

Deutscher Ausschuss fur Stahlbeton, Proceedings — Bundesallee 215/218, Room 302, Berlin, W 15, Germany

Deutscher Beton-Verein, Proceedings—Deutscher Beton-Verein E. V., Bahnhofstrasse 61, Postfach 627, Wiesbaden, Germany

Engineer, The—28 Essex St., Strand, London, WC 2, England

Engineering — 35—36 Bedford St., Strand, London, WC 2, England

Engineering Journal, The—2050 Mansfield St., Montreal 2, Quebec, Canada

Engineering News-Record — 330 West 42nd St., New York 36, N. Y.

Genie Civil, Le—Rue Jules Lefebvre, Paris 9e, France

Giornale del Genio Civile—Via Nomentana 2, Rome, Italy

Gradbeni Vestnik—Erjavceva 11a, Ljubljana, Yugoslavia

Highway Research Abstracts—Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Highway Research Board, Bulletin — Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Highway Research Board, Proceedings — Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Hungarian Technical Abstracts—Hungarian Central Technical Library, P. O. Box 12, Budapest 8, Hungary

In the Field of Building—Building Research Station, Technion—Israel Institute of Technology, Haifa, Israel

Indian Concrete Journal, The — 121 Queen's Road, Bombay 1, India

Industrial and Engineering Chemistry —American Chemical Society, 1155 Sixteenth St., NW, Washington 6, D. C.

Informes de la Construccion—Institute Técnico de la Construccion y del Cemento del Patronato Juan de la Cierva de Investigaciòn Tecnica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Ingenieria—Escuela Nacional de Ingenieria, Ciudad Univistaria, Mexico, D. F., Mexico

Ingenieria Civil—Assocacion de Ingenieros Civiles del Perú, Colmena 788, 40 Piso, Lima, Peru

Ingenioren — Dansk Ingeniorforening, Ingeniorhuset, 31 V. Farimagsgade, Copenhagen V, Denmark

Institution of Civil Engineers, Proceedings—Institution of Civil Engineers, Great George St., Westminster, London SW 1, England

Institution of Civil Engineers of Ireland, Transactions — Institution of Civil Engineers of Ireland, 35 Dawson St., Dublin, Ireland

Instituto Técnico de la Construccion y del Cemento, Bulletin—Instituto Técnico de la Construcciòn y del Cemento del Patronato Juan de la Cierva de Investigaciòn Técnica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Interbuild — 11 Manchester Square, London, W 1, England

International Civil Engineer and Contractor—Lennox House, Norfold St., London, WC 2, England

Iowa Engineering Experiment Station, Bulletin—Iowa Engineering Experiment Station, Iowa State University, Ames, Iowa

Japan Society of Civil Engineers, Transactions—Japan Society of Civil Engineers, 1-Chome, Yotsuya, Shinyuko-Ku, Tokyo, Japan

Journal of the Boston Society of Civil Engineers—Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Journal of the American Ceramic Society and Ceramic Abstracts—American Ceramic Society, 4055 N. High St., Columbus 14, Ohio

Journal of the Institution of Engineers, Australia — Institution of Engineers, Gloucester and Essex Sts., Sydney, N. S. W., Australia

Journal of the Japan Society of Civil Engineers — Japan Society of Civil Engineers, 1-Chome, Yotsuya, Shinyuko-Ku, Tokyo, Japan

Journal of the Prestressed Concrete Institute—207 W. Wacker Drive, Chicago, Ill.

Journal of Research—National Bureau of Standards, Washington 25, D. C.

Magazine of Concrete Research — Cement and Concrete Association, 52 Grosvenor Gardens, London, SW 1, England

Materiales Maquinaria y Metodos para la Construccion—Asociacion Tecnica de Derivados del Cemento, Balmes, 163, Barcelona, Spain

Memoires de la Société des Ingénieurs Civils de France—19 Rue Blanche, Paris 9e, France

Memoirs of the Faculty of Engineering Kyoto University—Kyoto University, Kyoto, Japan

Modern Concrete—431 S. Dearborn St., Chicago 5, Ill.

Nase Gradevinarstvo — Association of Societies of Civil Engineers and Technicians of Yugoslavia, Kneza Milosa 7, Belgrade, Yugoslavia

National Building Research Institute, Bulletin—South African Council for Scientific and Industrial Research, Pretoria, South Africa

New Zealand Concrete Construction—New Zealand Portland Cement Association, 102 Featherston St.,Wellington, C 1, New Zealand

Nordisk Betong—Nordisk Betonforbund (Nordic Concrete Association), Stockholm 70, Sweden

Novaia Tekhnika i Peredovoi Opyt v Stroitelstve—Moscow, Russia

Oesterreichische Ingenieur - Zeitschrift —Mölkerbastei 5, Vienna I, Austria

Ontario Hydro Research News — Research Division, Hydro-Electric Power Commission of Ontario, 620 University Ave., Toronto 2, Ont., Canada

Pit and Quarry—431 S. Dearborn St., Chicago 5, Ill.

Polish Technical Abstracts—Centralny Instytut Dokumentacji Naukowo-Technicznej, Aleja Niepodleglosci 188, Warsaw, 12, Poland

Polytechnisch Tijdschrift — 28 Balistraat, The Hague, Netherlands

Precontrainte - Prestressing — 21 Rue Newton, Brussels 4, Belgium

Progressive Architecture — 430 Park Ave., New York 22, N. Y.

Public Roads—Bureau of Public Roads, U. S. Department of Commerce, Washington 25, D.C.

Rakennusinsinoori — Liisankatu 18.D, 49, Helsinki, Finland

Reinforced Concrete Review, The—Reinforced Concrete Association, 94-98 Petty France, London, SW 1, England

Revista Constructiilor si a Materialelor de Constructii — Revistele Tehnice ASIT, Str. Ioan Ghica nr. 3, Bucharest, Rumania

Revista del Colegio de Ingenieros de Venezuela—Parque Los Caobos, Apartado 2.006, Caracas, Venezuela

Revue des Materiaux—Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, 19 Rue Lafayette, Paris 9e, France

Roads and Streets—24 W. Maple St., Chicago 10, Ill.

Road Research Notes—Road Research Laboratory, Harmondsworth, Middlesex, England

Rock Products—79 W. Monroe St., Chicago 3, Ill.

Schweizer Archiv für angewandte Wissenschaft und Teknik — c/o Buch-

druckerei Vogt-Schild AG, Solothurn, Switzerland

Statens Provningsanstalt, Meddellanden (Bulletins) — Statens Provningsanstalt Stockholm, Sweden

Stavebnicky Casopsis Slovenskej Akadémie Vied—Klemensova 27, Czechoslovakia

Structural Engineer, The — Institution of Structural Engineers, 11 Upper Belgrave St., London, SW 1, England

Swedish Cement and Concrete Research Institute Handlingar (Proceedings)—Svenska Forskningsinstitutet for Cement och Betong vid Kungl. Tekniska Hogskolan I, Stockholm, Sweden

Technical Translations—Office of Technical Services, Department of Commerce, Washington 25, D. C.

Travaux—6 Avenue Pierre 1er-de-Serbie, Paris 16e, France

University of Illinois Engineering Experiment Station, Bulletin—University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.

Vag-Och Vattenbyggaren — Brunkebergstorg 20, Stockholm C, Sweden

VDI Zeitschrift—Verein Deutscher Ingenieure-Verlag GmbH, Ingenieurhaus, Prinz-Georg-Strasse 77, Düsseldorf, Germany

Waterways Experiment Station, Technical Memorandum—Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

Zement-Kalk-Gips — Kleine Wilhelmstrasse 7, Wiesbaden 16, Germany

Zement und Beton—Oesterreichischen Betonvereins und Verein der Oesterreichischen Zementfabrikanten, Strohgasse No. 21 A, Vienna 3, Austria

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Bridges

Mangfall bridge

The Engineer (London), V. 209, No. 5423, Jan. 1, 1960, p. 32

Reviewed by ARON L. MIRSKY

Brief description of a rather unusual prestressed concrete bridge carrying the Munich-Salzburg *autobahn* over the valley of the Mangfall, near Weyarn, in Bavaria. The main structural elements are two double-cancellation lattice girders 6.61 m high with panels 5.91 m long. The top flange of each girder is formed by the prestressed concrete deck which cantilevers from either side of the girder, the two slabs being joined at the center by a central rib which forms a longitudinal divider between roadways. The lower flanges are also joined, to form a lower deck which will be utilized as a cycle and pedestrian path.

Construction procedures were also interesting. The existing temporary steel structure was moved laterally onto auxiliary piers; hollow concrete columns for these are 3 m in diameter, with 0.25 m thick walls, built with slip-forms.

The new structure is being built in 48 stages at a rate of 6 m per week, partly as a free cantilever (maximum cantilever, 58 m).

Suspension bridges in prestressed concrete (Hangebrucken aus vorgespanntem Beton)

D. VANDEPITTE, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 5, May 1959, pp. 156-164

Reviewed by FERDINAND S. ROSTASY

For a great ratio of dead load to live load, i.e., long spans, it is often more economical to place the prestressing tendon outside the cross section of the girder. This leads to a structure of the bowstring type or more reasonably to the suspension bridge. The author establishes the relationship to the ordinary steel suspension bridge and describes the criteria of design and analysis briefly. The most characteristic feature of the concrete suspension bridge is that the to be anchored cable

force can be used with advantage to precompress the girders of the roadway. Two bridges constructed in Holland are discussed in detail and the limits of the application and the ability to compete with steel suspension bridges are outlined.

Construction

Manufacture of cylindrical septic tanks (Fabrication de fosses septiques cylindriques)

P. Rebut, *Revue des Materiaux* (Paris), No. 528, Sept. 1959, pp. 218-220
Reviewed by Phillip L. Melville

Methods of manufacturing cylindrical tanks are reviewed. Use of vibration for parts is recommended with tanks being field assembled.

Dome structures

Consulting Engineer, V. 13, No. 6, Dec. 1959, pp. 89-122
Reviewed by Aron L. Mirsky

This report is divided into six parts: (1) A philsophy of space and shape, by R. Buckminster Fuller, pp. 90-96; (2) A reconstruction — contrast in technologies, pp. 97-98; (3) Engineering aspects, pp. 99-106; (4) Designs in metal, pp. 107-114; (5) Designs in concrete, pp. 115-119; and (6) Designs in wood and other materials, pp. 120-122.

In a relatively small space, it manages to cover domes of many types—what has been accomplished; the problems—structural, acoustical, electrical, mechanical—that faced (and face) the designers; the solutions.

The first part is difficult reading, both the terminology and the marginal sketches being very much in the spirit of Torroja's book.* While Fuller's geodesic tensegrity concept does not apply directly to the usual concrete dome it does apply to the Nervi type of lamellar structure.

The second part describes briefly the reconstruction of Saint Hedwig's Cathedral in East Berlin, originally a timber dome, reconstructed in precast concrete.

The third part covers in some detail the engineering problems encountered in the design and construction of domes. It is illustrated by numerous case histories.

The remaining three parts cover various completed structures, and are primarily of the case history type, supplementing the coverage of the third part.

In all, interesting—and instructive—reading.

*Torroja, Eduardo, *Philosophy of Structures*, University of California Press, 1958 [Current Reviews, ACI Journal, Vo. 30, No. 4, Oct. 1958 (*Proceedings* V. 55), pp. 519-520].

Dams

Field study of interior temperatures in concrete

D. L. Houghton, *Proceedings*, ASCE, V. 85, PO 5, Oct. 1959, pp. 21-43
Author's Summary

Data obtained on interior temperatures attained by mass concrete in two types of concrete dams is reported, one a thin arch structure, the other a massive gravity structure. Cement contents of the concrete investigated ranged from 2.0 to 4.75 sacks per cu yd, and average compressive strengths ranged from 1610 to 4285 psi at 28 days. The "coefficient of maximum temperature increase" based on the design cement content of the concrete investigated ranged from 6.1 to 15.3 deg F per sack of cement content, the mean coefficient being about 11 deg F per sack.

New dam techniques

Andre Coyne, *Proceedings*, Institution of Civil Engineers (London), V. 14 (Session 1959-60), Nov. 1959, pp. 275-290
Reviewed by Aron L. Mirsky

An acknowledged master discusses progress in arch dam design and construction, in the recent past and the forseeable future. Progress in three main types is analyzed: single-arch dams (in terms of stresses and shape), multiple-arch dams, and combined structures (particularly as exemplified by Djatiluhur Dam).

Second paragraph of paper is worth quoting in its entirety, both in its own

right and in view of certain recent events:

"There can be no rest for the designer who, in the interest of aesthetics, safety, and economy, seeks for better design and construction. The circumstances in which the designer of dams is placed constrain him to strive constantly for improvement. So he has to escape from slavish and constant adherence to precedent, and from exaggerated respect for "treatises" written with more haste than finesse. Precedents are helpful only in that they contribute towards enriching our experience. Each successful experiment is valuable. If it fails, its value is greater."

Design

Reinforced concrete folded plate construction

CHARLES S. WHITNEY, BOYD G. ANDERSON, and HAROLD BIRNBAUM, *Proceedings*, ASCE, V. 85, ST 8, Oct. 1959, pp. 15-43

AUTHOR'S SUMMARY

Presents design methods and discusses experience with the construction of reinforced concrete folded plate thin shells for the roofs and floors of hangars, auditoriums, warehouses, and other buildings. Quantities of materials and costs are compared with other types of structures.

Addendum to the static computation of folded plates (Zur statischen Berechnung der prismatischen Faltwerke)

G. GROSS, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 9, Sept. 1959, pp. 215-223

Reviewed by RUDOLPH SZILARD

A step by step method for computing stresses in folded plate structures has been introduced, taking into consideration the secondary stresses caused by the movements of joints. The first assumption is based on the membrane theory, i.e., the joints do not move, then one joint at a time is released and the effect of the joint displacement is computed. The diagram of the moments is based on a sinusoidal distribution (first term of a Fourier series); correction factors are given for other distribution diagrams. The final deflection is expressed as a linear function of the external load. Symmetrical and unsymmetrical loads can be treated.

Buckling

English translation of report of CEB Committee No. 8, Buckling, *Bulletin D'Information* No. 17, Comite Europeen du Beton, Vienna, Apr. 1959 (Limited supply of single copies available from ACI headquarters)

This is the first of a series of English translations of reports of the Comité Europeen du Béton being made available by the ACI-CEB Collaboration Committee.

This translation includes the report of CEB Committee No. 8, Buckling, containing its recommendations for the design of slender columns or arches and plates and shells in biaxial compression subject to buckling; Appendix 1, proposed method of buckling analysis; and Appendix 2, interpretation of 356 tests of eccentric compression and of 143 tests of buckling.

Calculations for spanned structures of prefabricated ferroconcrete bridges of the beam type, taking into account the elastic distribution of the load (in Russian)

B. P. NAZARENKO, Khar'kov Regional Scientific-Technical Conference Relating to Ferroconcrete Structures, Dec. 13-15, 1954; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 4, 1958, Rev. 4715

APPLIED MECHANICS REVIEWS Dec. 1959

Results are given for the calculations of a composite beam network under the action of a concentrated symmetrical force, with no account being taken of the torsion of the beams. In the proposed method of calculation for the beam system the distribution of the load into a transverse direction is performed by its transfer onto four to five longitudinal beams with three transverse beams; evidence for the soundness of the method is not forthcoming; there is a reference to agreement between the calculated and experimental data.

Pavements

Factors which affect the skid resistance of concrete

Concrete Construction, V. 5, No. 2, Feb. 1960, pp. 29-31

Discusses briefly some of the factors which affect the skid resistance of concrete pavements including: age and condition of the surface, type of aggregate and binder, the surface texture, climate and other factors which affect weathering, temperature, moisture, and the presence of foreign materials.

Electronics assist in highway construction

HAROLD HARRIS, *Electronics*, V. 32, No. 51, Dec. 18, 1959, pp. 69-71

Reviewed by ARON L. MIRSKY

This article, describing and picturing the various instruments developed for and used in the AASHO Road Test, amounts to a capsule catalog of what is available—and ends with a note to the effect that what is not available, but is badly needed, is a nondestructive system for makking accurate moisture content determinations in a 1-in. layer of soil immediately below the roadbed.

Thin bonded concrete surfacings applied to existing concrete road slabs

R. I. T. WILLIAMS, *Roads and Road Construction* (London), V. 37, No. 439, July 1959, pp. 194-201

HIGHWAY RESEARCH ABSTRACTS Jan. 1960

Describes a trial in which concrete surfacings varying in thickness from ⅜ to 3 in. were applied to existing concrete slabs. The preparation of the surface of the base concrete included areas of washing and brushing, surface roughening by power-driven machine, acid treatment, and, for the greater part of the work, the use of all these methods together.

With the exception of one area, the existing concrete slabs were between 4 and 8 years old, laid in bays approximately 10 ft square, and joints were formed in the surfacing to coincide with the joints in the base slab. One area, however, was a month old; this was 81 ft 3 in. long, with a single transverse joint near its midpoint. No joints were formed in the surfacing above this area but oblong mesh reinforcement weighing 7 lb per sq yd was provided in the surfacing.

Cores were extracted from 12 positions in the finished work 2 to 4 months after the surfacing was completed, all but one core being removed whole. The success with which cores were extracted suggests the bond developed is satisfactory but insufficient cores have so far been examined to reach firm conclusions regarding the influence of base texture, surface treatment, and thickness of surfacing on the bond developed.

Longitudinal distribution of wheel loads on a runway

R. W. SMITH and R. HORONJEFF, *Proceedings*, ASCE, V. 85, AT 4, Oct. 1959, pp. 1-14

AUTHORS' SUMMARY

As a result of the analysis presented, the following conclusions are felt to be valid:

1. The variation of aircraft loads in a londitudinal direction on a runway may be analyzed with sufficient accuracy to provide data for required runway thickness.

2. The rate of decrease in wheel loading of jet-powered aircraft is significantly slower than that for propellor-driven aircraft. This fact should be considered if criteria for variable runway thickness in a longitudinal direction are to be considered.

New slipform paver makes road as smooth as California wants it

Engineering News-Record, V. 164, No. 2, Jan. 14, 1960, pp. 38-40

An account of some of California's paving specifications. Major part of article describes the smoothness specification and a paver that meets it. Paver places up to 4000 ft of 24 ft wide 8 in. thick pavement per day.

The smoothness specification limits surface variation to ⅛ in. in 10 ft measured from a straightedge.

Precast Concrete

Vibrated concrete pipes (Tuyaux en beton moules sous vibration)

P. Rebut, *Revue des Materiaux* (Paris), No. 526-527, July-Aug. 1959, pp. 188-198
Reviewed by Phillip L. Melville

A study of vibration in the manufacture of plain, reinforced, and prestressed concrete pipe. The various systems are evaluated from the point of view of industrial production.

Tests on the manufacture and strength of small diameter concrete pipes made with sulfate cement (Essais de fabrication et de resistance de tuyaux de faible diametre en beton de ciment sursulfate)

L. Blondiau, *Revue des Materiaux* (Paris), No. 528, Sept. 1959, pp. 212-217
Reviewed by Phillip L. Melville

Pipe were rated on the basis of compressive and flexural strengths, absorption and watertightness, and durability. The use of sulfate cement was found satisfactory.

Suspension roof of the large training and exhibition hall of the Westfalenhalle Co. in Dortmund (Das Hangedach der grossen Trainings—und Ausstellungshalle der Westfalenhalle A.G. in Dortmund)

F. Vaessen, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 10, Oct. 1959, pp. 233-240
Reviewed by Rudolph Szilard

Describes a remarkable utilization of prestressing and precasting for economically spanning, without internal support, an 80 m span of an exhibition hall built in Germany. The parabolic suspension-roof construction has 5.0 m sag at the center line of the building. The prefabricated ribs consist of 12 x 22-cm reinforced elements, spaced 1.2 m center to center, and are connected by means of prestressing strands (post-tensioning method). The prestressed ribs, with lightweight prefabricated elements between them, are joined by means of grouting and form a monolithic roof structure. The prestressed cables are supported by reinforced concrete frameworks and are anchored deep in the earth utilizing passive earth pressure. The action of the forces, construction details, and construction methods are discussed. The critical wind loads have been determined by means of windtunnel tests.

Prestressed Concrete

Radistres — A new idea in prestressed panels

Concrete, V. 68, No. 1, Jan. 1960, pp. 16-19

A condensed report on the development and methods to date in the Radistres method, a method for radial prestressing of concrete panels. Instead of being a linear one-direction form of prestressing, the radial prestressing uses a method of placing continuous bands under tension. The stressing machine is shaped like a large X. Fingers on the X extend downward into the form. The bands are placed under tension, held by these fingers, and then the concrete is placed in the form. After the concrete has set sufficiently, the fingers are withdrawn and the holes filled.

Design of prestressed composite steel structures

Rudolph Szilard, *Proceedings*, ASCE, V. 85, ST 9, Nov. 1959, pp. 97-123
Author's Summary

Design and construction problems of statically determinate prestressed composite structures (concrete slab and steel beams) are treated. After review of the properties of the materials, different construction methods are discussed, including approximate and more exact methods for determination of prestress loss. Elastic and plastic design methods are derived.

The paper deals with the design equations covering the problems connected with prestressing composite steel structures. Part 1 treats girders having statically determinate external

supports. After reviewing the exact physical properties of the materials used, the three basic construction methods are discussed. The computed initial prestressing force is reduced by a number of factors. To determine the loss of prestressing force, approximate and more exact methods are given. Ultimate load design and the economical advantages of the prestressed composite structures are treated.

About the dynamic modulus of elasticity of prestressed concrete (Uber der dynamischen E-Modul von Spannbeton)

F. P. Muller, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 8, Aug. 1959, pp. 192-197
Reviewed by Rudolph Szilard

To determine the effect of dynamic loads on prestressed concrete structures, the knowledge of an exact dynamic modulus of elasticity is of importance.

The prestressed concrete test specimens were supported at the node-points of the characteristic shape of the vibration. Test results indicate that a small nonlinearity exists beetween the stress and strain. The effect of the magnitude of prestressing and the absolute magnitude of the amplitude is negligible. There is no important difference between dynamic F of prestressed and that of conventional reinforced concrete.

Practical prestressed concrete

H. Kent Preston, McGraw-Hill Book Co., Inc., New York, 1960, 340 pp., $11.50

A useful book to introduce a structural designer to the principles and applications of prestressed concrete when he has not had benefit of previous indoctrination. Beginning with simple principles of plain and reinforced concrete, the author proceeds to those properties peculiar to prestressed concrete. After outlining the steps in design procedure, he presents a numerical example to demonstrate the mathematics involved in the design of a typical prestressed concrete member. He describes the materials used and the methods of pretensioning and post-tensioning the reinforcement.

One chapter is devoted to the tentative recommendations for prestressed concrete developed by ACI-ASCE Committee 323. The appendix reprints the reader and committee comments on this report.

The author goes through the designs of a prestressed concrete bridge, a double-T roof in lightweight concrete, and a post-tensioned girder. He discusses in detail prestressed piles, typical members, and continuous structures, and concludes with practical considerations and limitations.

The appendix also includes properties and specifications for wires or tendons and tentative specifications of the Prestressed Concrete Institute and the Bureau of Public Roads.

Structural Research

Analysis of tests in pure flexure and eccentric compression (in Spanish)

E. Torroja and J. M. Urcelay, *Bulletin* No. 201, Instituto Técnico de la construccion y del Cemento (Madrid), 1959, 27 pp.
Reviewed by Emilio Rosenblueth

Statistical analysis of 234 tests carried out in Europe and the United States on reinforced concrete members of rectangular cross section. It is shown that optimum adjustment with assumption of uniform stress distribution in the compression zone is obtained when the concrete is assigned an ultimate unit strain of 0.0035. The average stress is taken as $0.9\ f_c'$, and the rectangular stress block covers 0.8 of the depth to the neutral axis. The corresponding coefficient of variation is 7.19 percent. The result applies to values of q greater than 0.2. When $q = 0.2$, the relation $M/Kbd^2 = 0.03 + 0.75q$ fits the test results satisfactorily. One interesting aspect of the work is the treatment of systematic tendencies that differ with the research program.

Behavior of fiber-reinforced plastic in flexure (Das Verhalten faserstoff-verstarkter Kunststoffe bei Biegebeanspruchung)

FRANZ BEER, *V D I Zeitschrift* (Düsseldorf), V. 101, No. 22, Aug. 1, 1959, pp. 1045-1050
Reviewed by ARON L. MIRSKY

In the case of structural elements reinforced with fibers all running parallel to the axis of the element, shear may be critical. A simple method of calculating this shear stress is developed, using the energy-of-distortion theory of failure. Additional reinforcement in the form of transversely-placed fibers is recommended.*

[This article is noted here for two reasons: first, because reinforced plastics and reinforced concrete have certain attributes in common, and second, because reinforced plastics can be, and have been, used in research (especially photoelastic) on reinforced concrete.]

*cf F. Beér, "Die Festigkeitseigenschaften kreuzweise bewehrter Kunststoffe," *VDI Zeitschrift*, V. 101, No. 12, Apr. 21, 1959, pp. 463-468; A. L. Donaldson and R. B. Velleu, "Design and Fabrication with Directionally Reinforced Plastics," *Modern Plastics*, V. 35, No. 2, Oct. 1957, pp. 133-135, 138, 140, 142, 279; and No. 3, Nov. 1957, pp. 143-144, 264, 267 (abstracted by F. Beér in *VDI Zeitschrift*, Aug. 1, 1959, pp. 1051-1052).

Direct-indicating polarization-optical measuring elements for analysis of strain and stress of structural elements (Unmittelbar anzeigende polarisationsoptische Messelemente fur die Dehnungs- und Spannungsanalyse an Bauteilen)

GEORG U. OPPEL, *VDI Zeitschrift* (Düsseldorf), V. 101, No. 20, July 11, 1959, pp. 809-816
Reviewed by ARON L. MIRSKY

Describes two newly-developed photoelastic gages (using basic method described by author in *VDI Zeitschrift*, V. 81, No. 27, 1937, pp. 803-804), one for direct indication of strains, the other for direct indication of the orientation of principal strains or principal stresses. High sensitivity is attained by making use of phase overlap (interference bands) and of notch effect. Devices obviate recourse to models, are simple to apply and use, permit simultaneous observation of strains at several points, and can be used with both static and dynamic loads.

Comparison of the gages with those sold in America under the name "Photostress" (not referred to by author) will prove of interest.

General

Structural concrete hospitals

Cement and Concrete Association, London, 1957, 25 pp., $1.50
BRI ABSTRACTS
June 1959

This booklet carries 40 photographs of hospitals, some with floor plans, and cites a statement by a member of the engineering faculty of London University to the effect that reinforced concrete is now generally favored abroad on economic grounds, in preference to other building materials, for building frames up to 25 stories in height. Examples of hospital construction are shown from Finland, Israel, Sweden, Switzerland, Holland, Brazil, Japan, Ghana, Nigeria, and India.

Gravel prospecting by use of aerial photographic interpretation

R. J. KASPER, *Bulletin* 213, Highway Research Board, Jan. 1958, pp. 44-52

The rapidly diminishing aggregate supply has made it imperative to expedite prospecting for gravel. This was and is being done by combining the sciences of geology, aerial photographic interpretation, and geophysics into a research unit whose purpose is the reconnaissane and exploration of economically situated sand and gravel deposits.

Removal of stains from concrete

D. HARRISON, *Ontario Hydro Research News*, Hydro-Electric Power Commission of Ontario (Toronto), V. 11, No. 3, July-Sept. 1959, pp. 19-24

Accidental spillage on concrete structures can create difficult problems of stain removal. To provide some guidance to field personnel and architects on techniques for removing stains from

concrete, a number of recommended methods were evaluated through a series of laboratory tests.

Materials used in stain removal tests are described and precautions necessary in the use of various solvents and techniques are emphasized. Stain removal procedures are described for copper, iron or rust, linseed oil, petroleum oils, grease, asphalts, wood tar and smoke, creosote, paint, and inks.

Locating reinforcing bars in structures using nondestructive methods (in Finnish)

S. E. PIHLAJAVAARA, *Rakennusinsinööri* (Helsinki), V. 16, No. 2, pp. 40-45
Reviewed by BEATO KELOPUU

Describes briefly the use of two nondestructive methods for locating reinforcing bars in hardened concrete. The two methods are the application of electromagnetic instruments, or so-called covermeters, and the use of gammaradiography. The results obtained indicate that the methods are useful and inexpensive ways of observing concealed features in reinforced concrete. The radiographs obtained on some sites showed that the reinforcing bars were far from their right position.

Proceedings of the symposium on the failure and defects of bridges and structures

Japan Society for the Promotion of Science, Tokyo, Dec. 1958, 110 pp.

Contains ten papers presented at a symposium held by the National Committee for Bridge and Structural Engineering of the Science Council of Japan in cooperation with Japan Society of Civil Engineers and Architectural Institute of Japan. The reports cover damage to and correction of reinforced concrete and steel structures.

Papers concerned with concrete structures were: "Counteraction Against the Damage to Buildings Caused by Defects in Waterproofing Methods;" "Damage to Bridge Structure for Elevated Track in Osaka Station and the Countermeasures Taken;" "On the Cracks in Some Reinforced Concrete Bridges;" "On the Researches of Concrete Shrinkage and Cracks in Setting, Hardening, and Drying (Especially in Reinforced Concrete Building);" "Effect of Structural Reinforcing on Vibrational Behaviors of a Fire-Damaged Reinforced Concrete Building;" and "Cause of Damage and Effect of Strengthening Methods at the Kugenuma Bridge."

The architect and the engineer

EDMUND D. J. MATHEWS, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59, pp. 499-502

The engineer and the architect

OVE N. ARUP, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), Aug. 1959, pp. 503-510
Reviewed by ARON L. MIRSKY

Contrary to the stand taken by Michael Ryan in discussing the integrated Heinz plant at Kitt Green, Wigan, *, † the emphasis here is on *collaboration*. Mr. Mathews, after noting that the architect and the engineer both have their peculiar fields of competence, noted further that there was considerable overlapping of fields, and that there was need for unifying education. Mr. Arup, making similar observations, stressed the proliferation of knowledge which logically leads to the collaboration of specialists—*not* rivalry. Among other interesting thoughts, he notes the advantages of giving the client a chance at selection from several preliminary designs, the results either of multiple studies by one designer or of a design competition.

The discussion (pp. 510-532 of the same issue) is both spirited and valuable.

*"Architecture to Reflect a Process—Flow Line Food Production in a Factory Built for Heinz," *Engineering* (London), V. 187, No. 4860, May 1, 1959, pp. 576-578; Current Reviews, ACI JOURNAL, V. 31, No. 4, Oct. 1959 (*Proceedings* V. 56), p. 360.

†and to the stand taken by the architects in the current McCamy case in New Jersey (*American Engineer*, V. 29, No. 12, Dec. 1959, pp. 31-32).

DISCUSSION, PROCEEDINGS V. 56

Discussion of papers published in the October through December, 1959, JOURNALS appears in the concluding pages of this June issue, as the Institute continues its quarterly publication of discussion. Discussion of papers published January through March, 1960, will appear in Part 2, September 1960 ACI JOURNAL. Discussion of April through June, 1960, papers will appear in Part 2, December 1960 JOURNAL along with index and errata for V. 56.

ACI NEWS LETTER

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Technical Committees

Backbone of ACI Progress

Publication of the annual roster of technical committees and their members on the following pages is a reminder of the contribution to and influence on the course of ACI affairs of this small group of dedicated individuals. Less than 10 percent—it should be more—of the Institute membership devotes the extra time, energy, and talent necessary to the development of authoritative technical information making up ACI technical committee reports and Institute standards. This outstanding record of achievement is the more remarkable because it is all volunteered work even involving, in some cases, expense on the committee member's part to fulfill his particular committee assignment. Yet there are some who make similar contributions to the work of two or more ACI technical committees.

52 committees in action

The diversity and number of subjects studied by technical committees have increased materially in recent years. Twenty-eight technical committees in 1949 have expanded to the 52 listed here. The pace and depth of committee work has grown. Technical committees now claim an exclusive day and one-half at the annual meeting.

If you have a particular talent or specialized knowledge which you wish to apply to ACI committee work, let headquarters know about it. There may be a place now or in the near future where it can be effectively utilized.

ACI Technical Committees

Committee rosters as of May 1, 1960**

Technical Activities Committee
Organized 1947

The Technical Activities Committee is responsible, under the Board of Direction, for technical publications, convention programs, and technical committee activities (see p. 26, *1959 ACI Directory*). Terms of members expire at the annual convention of the year indicated. The President of the Institute is a member during and by reason of his tenure of office; similarly the Secretary-Treasurer of the Institute is secretary of the committee. The chairman, president, and secretary constitute the committee's executive group.

CLYDE E. KESLER (1961) Chairman
JOE W. KELLY (Ex Officio)
WILLIAM A. MAPLES, Secretary
G. E. BURNETT (1962)
HARRY ELLSBERG (1962)
E. A. FINNEY (1962)
BRYANT MATHER (1961)
WALTER J. MCCOY (1961)
J. J. SHIDELER (1961)

Standards Committee
Organized 1937

The Standards Committee supervises the promulgation of standards according to rules adopted by the Board of Direction (see p. 26, *1959 ACI Directory*).

DOUGLAS MCHENRY (1961) Chairman
BRUCE E. FOSTER (1962)
THOR GERMUNDSSON (1963)
JOE W. KELLY (1961)
THOMAS B. KENNEDY (1962)
JAMES A. MCCARTHY (1961)
WALTER J. MCCOY (1963)

Committee 115—Research
Organized 1921

Mission—This committee was organized to affiliate the interests of research workers and agencies in the field of concrete. Its assignment is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives.

Activities—A compilation of research projects is prepared and distributed each year at the open session of the committee at the annual convention.

The 1960 compilation lists 115 organizations and over 800 projects in progress. The compilation is divided into three sections: educational institutions in the United States, noneducational institutions in the United States, and institutions outside the United States.

Executive Group

CHESTER P. SIESS, Chairman
University of Illinois
RAYMOND E. DAVIS, Vice-Chairman
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Committee 116—Nomenclature
Organized 1957

Mission—Committee 116 is responsible for maintaining an up-to-date glossary of terms in the field of cement and concrete technology. The committee will recommend standard terminology for materials, conditions, and practices for which there is now a confusion of terms. Liaison will be maintained with other organizations in more specialized segments of the field to assist in the determination of generally accepted usage. As a standing mission, the committee will solve problems in terminology created by new developments for periodic additions to maintain the glossary current.

Activities—It is the intention to develop as comprehensive a glossary as practicable, limited to terms used in the field of cement and concrete. The initial report may be restricted to terms in common usage including those generally accepted and those for which usage varies. Where a confusion in usage exists, the committee function is to resolve it by recommending a rational definition of, and distinction among, terms. Subsequent revisions of the report will include solution of problems in terminology for new developments to prevent any increase in confusion of terms. Liaison with specialized organizations in the field will serve to define the scope of activity for Committee 116 and simultaneously to avoid conflict with recommendations developed by other organizations.

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Committee 201—Durability of Concrete in Service

Organized 1957

Mission—The committee is to review research on durability of concrete and past and present recommendations to insure satisfactorily durable concrete. It will report recommendations for materials and methods to secure concrete with maximum resistance to deterioration, to preserve concrete against deterioration, and to restore deteriorated concrete. The report may consist of suggested specifications or recommended practices to achieve the above objectives under conditions of exposure to various agencies of deterioration for common structures and pavements.

Activities—Included in the subject of durability will be resistance to abrasion, freezing and thawing, ice removal agents combined with freezing and thawing, and destructive (chemical) agents, but not erosion in hydraulic structures nor fire resistance. The report is not intended to present research results, which may be sufficient to justify a separate bibliography report. Discussion of research results leading to recommendations may precede recommendations if not available in bibliography references. The scope will be limited to material suitable for specifications or a basis for specifications; each division of the subject is to be treated individually in the most appropriate manner.

The committee sponsored a symposium on restoration of concrete at the 56th annual convention, March, 1960.

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Los Angeles Harbor Department

Committee 207—Properties of Mass Concrete

Organized 1930

Mission—Committee 207 is responsible for reporting developments in mass concrete construction. One main objective is to consider and report on the long-time behavior of mass concrete structures in service.

Activities—The inspection of mass concrete structures is a continuing project, with a mass of data being compiled. It is expected that these data will prove useful in preparing a report on performance of mass concrete. An attempt is being made to prepare an index on dams in which pertinent facts of construction methods, materials, exposures, performance, and other data would be catalogued.

The committee sponsored a session on the St. Lawrence seaway and power projects at the 54th annual convention in 1958.

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Committee 208—Bond Stress

Organized 1930

Mission—This committee is assigned to the promotion of research and to the collection and interpretation of data from field and laboratory to determine proper working stresses for bond in relation to type of bar employed in reinforced concrete construction. With regard to design matters the role of Committee 208 is purely advisory.

Activities—The principal activity of the committee during the recent past related to the ACI standard "Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)" which was published in the July, 1958 JOURNAL. It was approved as an ACI Standard at the 54th annual convention and ratified by letter ballot.

A subcommittee was appointed to work up a suggested program of research on bond, with special emphasis on splices, anchorages, development length, and cut-off bars, as well as the relation between bond and shear failure. Future work of the committee will depend in part on the report of this subcommittee. The committee will also focus its attention on those questions regarding bond that must be answered in connection with ultimate strength design and, eventually, with limit design.

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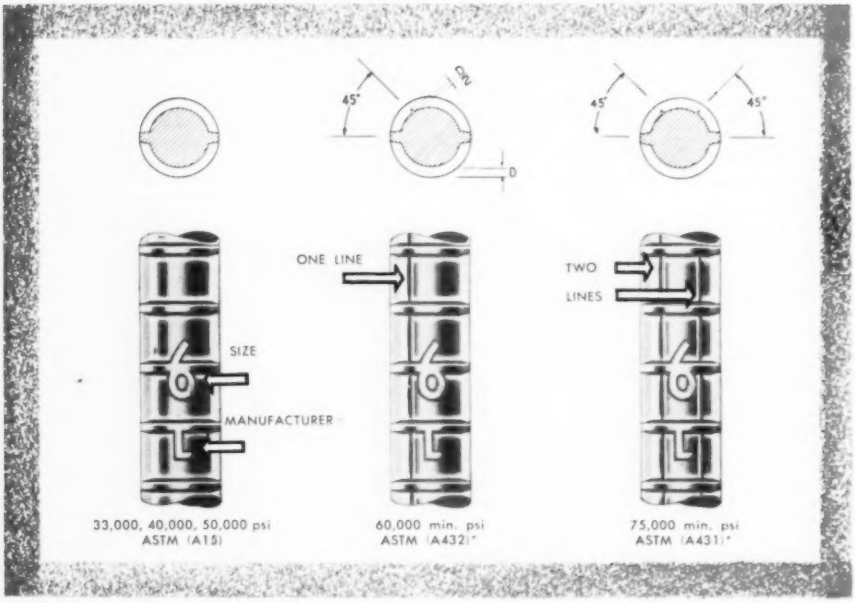

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Committee 209—Creep and Volume Changes in Concrete
Organized 1930

Mission—The assignment of Committee 209 is to review critically available data on the factors affecting the magnitude of creep and volume changes of concrete; to suggest or recommend topics of needed research; to stimulate the preparation of papers; and to interpret available information in a form that is useful to the designer.

This committee was reorganized in 1946 to consolidate the previous assignments of Committee 102, Volume Changes in Concrete, and Committee 109, Plastic Flow. Since that time the advent of prestressed concrete has introduced problems which were not anticipated in 1946, and the committee was again reorganized in 1953.

Activities—The committee sponsored a session on shrinkage of concrete at the 52nd annual convention in 1956; five papers were published in the February, 1957, JOURNAL.

Committee members are actively engaged in research in this field with the view toward publication of data useful to the structural designer. A bibliography on "creep in concrete" is also being prepared.

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Committee 210—Resistance to Erosion in Hydraulic Structures
Organized 1940
Discharged 1959

The committee report "Erosion Resistance of Concrete in Hydraulic Structures" was published in the November, 1955, JOURNAL (*Proceedings* V. 52). The report is concerned mainly with the physical erosion of concrete in hydraulic structures resulting from particles carried by flowing water and from pitting resulting from cavities forming and collapsing in water flowing at high velocities. Disintegration of concrete by chemical attack is also discussed. Materials, mix proportions, and construction procedures which will make concrete more resistant to such erosion are covered.

Committee 212—Admixtures
Organized 1943

Mission—This committee studies, assembles, and reports information on the effect of various admixtures, including air-entraining agents, on the properties of concrete. An admixture is defined as "a material other than water, aggregates, and portland cement (including air-entraining portland cement and portland blast-furnace cement) that is used as an ingredient of concrete and is added to the batch immediately before or during its mixing."

Activities—The committee's latest report, "Admixtures for Concrete," October, 1954, ACI JOURNAL, reported on the advantages and limitations of various types of admixtures.

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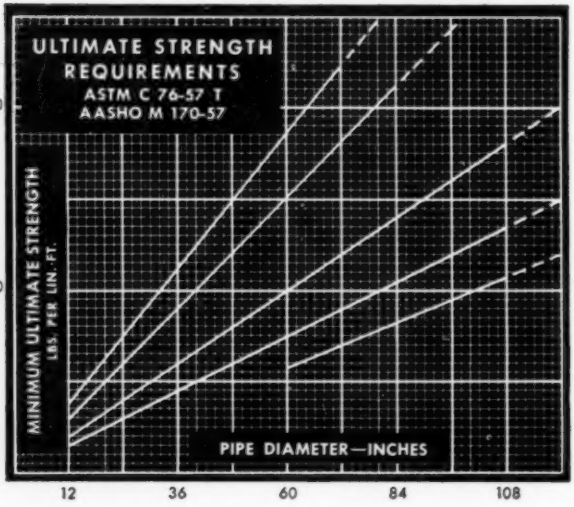

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Committee 213—Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

Organized 1946

Mission — This committee's assignment is to gather, correlate, and report available information on the properties of concrete made with mineral lightweight aggregates in the fields of monolithic structural concrete, precast concrete, and insulating and roof and floor fill concrete. A secondary mission of the committee is to report on design and construction practices in the field of lightweight concretes.

Activities—There are three subcommittees working on various phases of this assignment: (1) structural lightweight concrete; (2) lightweight concrete masonry; and (3) insulating concrete.

The committee also sponsored a symposium on lightweight aggregates and lightweight concrete at the 1955 ACI convention. The four reports were published in the October and November, 1956, JOURNALS.

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Committee 214—Evaluation of Results of Strength Tests of Concrete

Organized 1946

Mission — The committee's assignment is to analyze the variations which occur in the strength of concrete, to present tools of statistics which are useful in interpretation of these variations, and to discuss measures of reliability which can be used in establishing specifications and design criteria for the strength of concrete.

Activities—This committee produced the ACI Standard "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI

214-57)," which was published in the July, 1957, JOURNAL.

An annotated bibliography was published in 1960 as part of the ACI bibliography series.

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Committee 215—Fatigue of Concrete
Organized 1947

Mission—The assignment of this committee is threefold: (1) to review the available data on the behavior of plain and reinforced concrete under repeated loading, and to prepare a report on the present state of knowledge in the field; (2) to consider the implications of this knowledge in the design of members and structures of plain and reinforced concrete; and (3) to determine the gaps in our knowledge, and to recommend and encourage research which will provide the information to fill these gaps.

Activities—A final draft of an annotated bibliography on fatigue of concrete is in preparation with a view toward publication in the near future.

The committee also sponsored a session on fatigue of concrete at the 54th annual convention. The five papers of that symposium were published in the August, 1958, JOURNAL.

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Carnegie Institute of Technology

Committee 216—Fireproofing or Fire Protection of Structures
Organized 1955

Mission—This committee will study available data with a view to recommending proper ratings in concrete structures to make them more consistent and more in line with ratings given structures of other materials.

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Committee 312—Plain and Reinforced Concrete Arches
Organized 1930

Mission—The original mission was to review and correlate all information on arch design. This mission was later

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modified to include the preparation of a recommended practice or design manual.

Activities—This committee has published three reports in the ACI JOURNAL on reinforced concrete arch design—correlating much that has been learned of the influence of dead and live loads, and volume changes due to shrinkage, plastic flow, and temperature change. The most recent report, May, 1951, JOURNAL, presented a general design method, and a final report is in preparation including application of the ultimate strength method of design and the latest recommendations on volume change. (The previous two reports appeared in the March, 1932, and September, 1940, JOURNALS.)

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Association of American Railroads†
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Massachusetts State Highway Department

Committee 314—Rigid Frames for Buildings and Bridges

Organized 1935

Mission—The assignment of this committee is to study reinforced concrete and prestressed concrete rigid frames with particular reference to various loadings and the effects of volume changes resulting from shrinkage, creep, and temperature change. Included in this scope of the work are theoretical and experimental research, and investigation of actual bridges and building frames.

Activities—The change in scope and the change in committee title from "Rigid Frame Bridges" to "Rigid Frames for Buildings and Bridges" was approved by the Board of Direction in November, 1957. Work during the past year has been on a $\frac{1}{10}$-scale reinforced concrete model of an existing skewed rigid frame bridge and the development of a mathematical analysis for such structures. Previous work of Committee 314 has dealt mainly with tests on model beams and cylinders, on an aluminum model of a skewed-slab rigid frame bridge, and on a $\frac{1}{8}$-scale model of a skewed slab geometrically similar to the $\frac{1}{2}$-scale model tested previously at the University of Illinois. A report on this research, "Model Analysis of a Skewed Rigid Frame Bridge and Slab," by D. H. Pletta and Dan Frederick, was published in the November, 1954, ACI JOURNAL. A more recent report was given at the 56th annual convention, March, 1960.

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Committee 315—Detailing Reinforced Concrete Structures

Organized 1936

Mission—The committee's assignment is to prepare a report on detailing of reinforced concrete structures.

Activities—Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57) was published in 1957.

The manual presents recommended methods and standards for preparing drawings for the fabrication and placing

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of reinforcing steel in reinforced concrete structures. The previous ACI Standards ACI 315-51 (detailing of building structures) and ACI 315A-53 (detailing of highway structures) have been combined into one manual. Typical engineering drawings (and, for buildings, placing drawings as well) illustrate the use of the standards. There is also a chapter of instructions to designers.

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Committee 317—Reinforced Concrete Design Handbook
Organized 1939
Discharged 1959

The committee was reorganized in 1953 for the purpose of bringing up to date the ACI special publication *Reinforced Concrete Design Handbook* published in 1939. The second edition of the handbook, revised to conform with latest codes and practice, was made available late in 1955.

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†Corporation Member.
‡Contributing Member.

Committee 318—Standard Building Code
Organized 1929

Mission—The committee's assignment is to review periodically and up-date the Institute's "Building Code Requirements for Reinforced Concrete."

Activities—The latest edition, ACI 318-56, was adopted at the 1956 annual convention and ratified by letter ballot. It was published in the May, 1956, ACI Journal (*Proceedings* V. 52).

In 1957, work of the committee was devoted to two items: (1) The preparation of a question-and-answer paper explaining application of the appendix on ultimate strength design in ACI 318-56 (ACI Journal, Sept. 1957, *Proceedings* V. 54, p. 197). "Review of Changes in the ACI Building Code Requirements for Reinforced Concrete," by Frank Kerekes, formerly committee chairman, preceded the committee report in the September, 1957, Journal. (2) Developing a program for the next revision, including the creation of 15 subcommittees.

At the 11th ACI regional meeting in October, 1958, the committee sponsored a technical session devoted to changing design practices, the influence of new materials and design philosophy, and the relationship of these shifting ideas to impending changes in the ACI Code.

The committee will continue its standing assignment to improve the Code and provide safe minimum requirements for application of new methods of analysis, design, or construction and use of new materials.

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Committee 319—Recommended Practice for Use of Metal Supports for Reinforcement

Organized 1930
Discharged 1942

The report of this committee, "Recommended Practice for Use of Metal Supports for Reinforcement (ACI 319-

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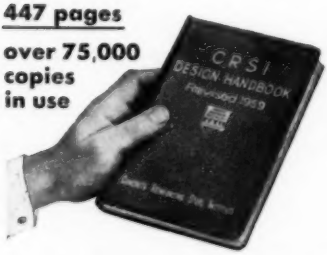

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42)," (June, 1945, ACI JOURNAL) was dropped as an ACI standard Oct. 30, 1955. The current material contained in the report is now covered by ACI 315-57.

Committee 320—Safe Loadings for Existing Concrete Bridges

Organized 1940
Discharged 1948

Committee issued a report, "Safe Loads for Existing Concrete Bridge," by F. W. Panhorst, author-chairman, January, 1943, ACI JOURNAL.

Committee 321—Design of Reinforced Concrete Slabs—Joint ACI-ASCE

Organized 1941

Mission—This committee, joined as an ACI-ASCE endeavor in February, 1953, is assigned the problem of analysis and design of bridge and building slabs.

Activities—The immediate objective is the problem of floor slabs in buildings, including the effect of openings and concentrated loadings. The committee is making a study of accurate solutions of the problem that have been developed in recent years, as well as of experimental data that have been made available; this with a view toward investigating the adequacy of current and proposed approximate methods of design.

Recent activity has been the preparation of a research program on the relative strength and behavior of different types of floor slab construction.

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American Iron and Steel Institute

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Committee 322—Design of Structural Plain Concrete

Organized 1942

Mission—Committee will review the 1942 report of the original committee, revise it, and expand it as necessary to provide a complete recommendation for the design of common unreinforced concrete applications. The ultimate objective is to develop a standard to supplement the "Building Code Requirements for Reinforced Concrete."

Activities—Reactivated in 1957, the proposed committee recommendations will include all subjects covered in the original report, "Proposed Recommended Stresses for Unreinforced Concrete," ACI JOURNAL, Nov. 1942, *Proceedings* V. 39. It may be expanded to include as new sections, design requirements for plain concrete walls, plain concrete underground structures, basements, and thin precast elements, and to provide more completely allowable loads for unreinforced piers, and temperature and shrinkage reinforcement required for thick slabs, thick walls, and slabs on ground.

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Committee 323—Prestressed Reinforced Concrete—Joint ACI-ASCE

Organized 1942

Mission—To continuously evaluate the technical status of prestressed concrete and to recommend to the joint sponsors the formation of any needed technical committees or subcommittees

for the study of particular developments, innovations, or problems arising in this field. Such committee activity should be directed toward the publishing of information and the development of tentative recommendations or recommended practices which incorporate new procedures or correct existing deficiencies. The committee will also study significant research problems and make recommendations.

Activities—Just reorganized under its new mission, the committee is still in process of reorganization. It was earlier reorganized as a joint ACI-ASCE group in 1952.

"Tentative Recommendations for Prestressed Concrete" was published in the January, 1958, JOURNAL. It constitutes a recommended practice, not a building code or specification. It is confined in scope to structural members involving linear prestressing. Most of the recommendations relate to flexural members and are intended to apply to both buildings and bridges.

A subcommittee has collected a bibliography consisting of over 2100 references, published from 1896 into 1955. A separate section lists patents. The second edition of *Bibliography on Prestressed Concrete* was published by ACI in 1955.

A subcommittee reviewed the patent situation on prestressed concrete, and prepared a paper, "Patents and Codes Relating to Prestressed Concrete," ACI JOURNAL, May, 1960.

A subcommittee prepared a set of definitions and notations, "Proposed Definitions and Notations for Prestressed Concrete," ACI JOURNAL, October, 1952.

The committee also sponsored a session at the 52nd annual convention in 1956 and the 53rd annual convention in 1957.

MORRIS SCHUPACK, Chairman
Schupack and Zollman

Committee 324—Precast Reinforced Concrete, Thin Sections

Organized 1946

Mission—The mission of this committee is to prepare recommendations for design and construction utilizing thin section precast concrete.

Activities—The committee completed its preliminary assignment of a comprehensive survey of available information dealing with the design and construction techniques of thin section precast concrete structures. Utilizing the compiled data of the survey, the committee has submitted a report, "Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction," which was published in the May, 1958, JOURNAL.

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Committee 325—Structural Design of Concrete Pavements for Highways and Airports

Organized 1948

Mission—The committee's assignment is to develop a recommended practice for the structural design of concrete highway and airport pavement — slab dimensions, cross section, joints, reinforcement, etc. — on the basis of the best information available, and to keep such recommendations up to date as new information indicates need for revision. These design recommendations

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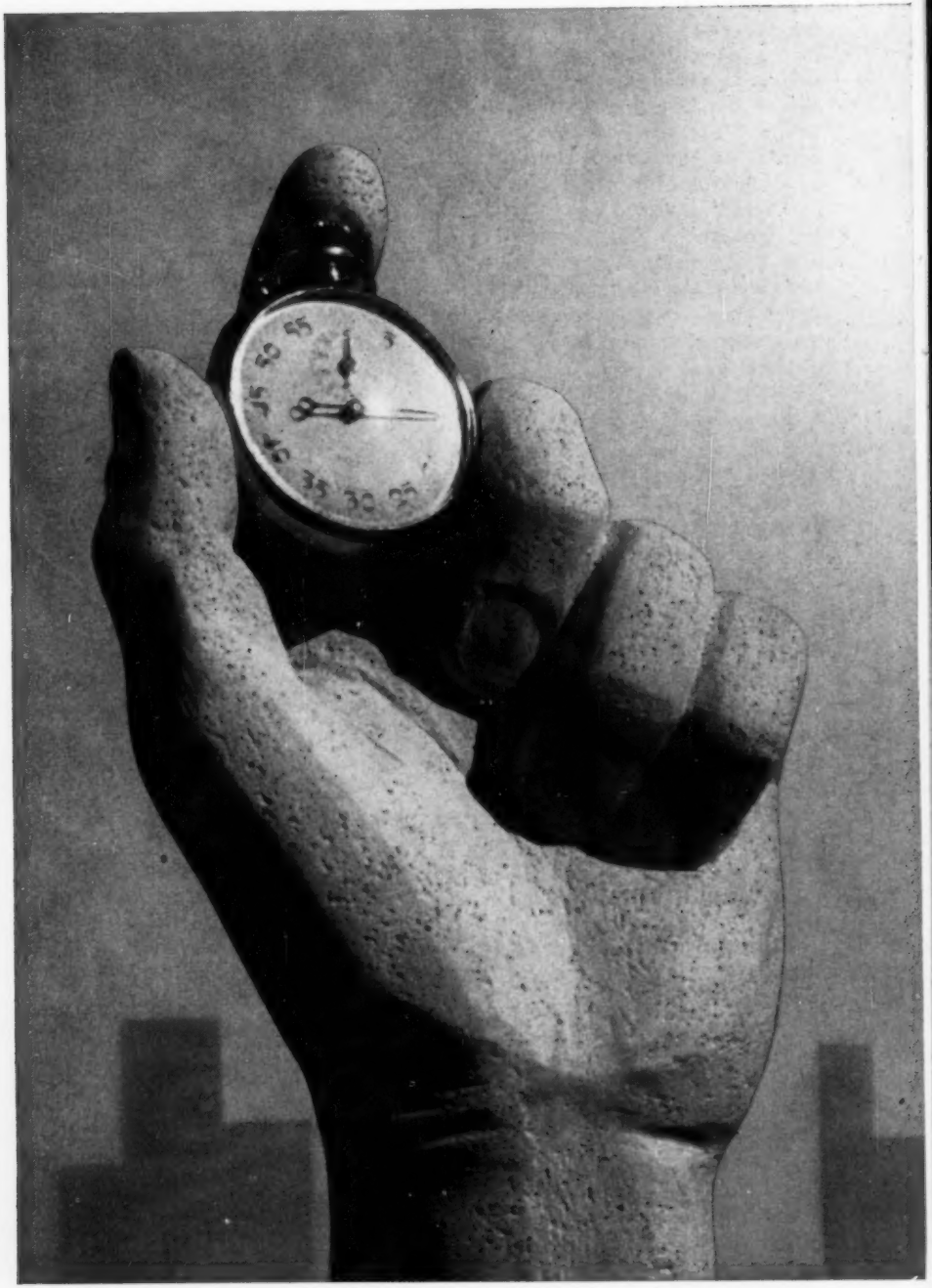

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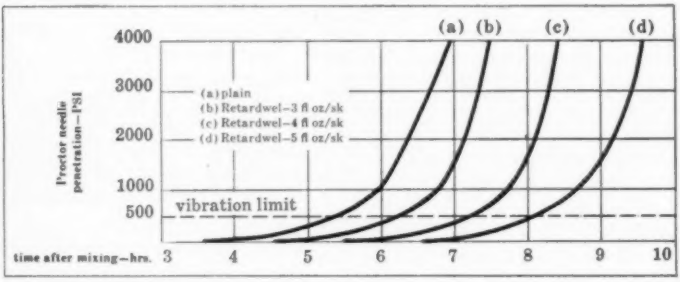

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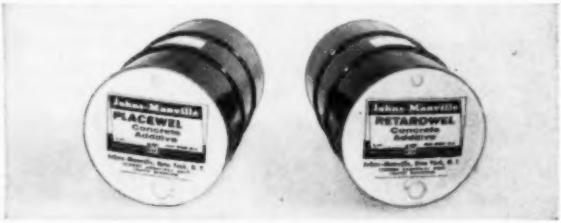

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complement the specifications for materials and construction prepared by Committee 617.

Activities—The ACI Standard "Recommended Practice for Design of Concrete Pavements (ACI 325-58)" was adopted at the 54th annual convention, ratified by letter ballot, and published in the July, 1958, JOURNAL.

Reports of the four subcommittees were published in 1956: Subcommittee I—"Considerations for Construction of Subgrades and Subbases for Rigid Pavement," August, 1956, JOURNAL; Subcommittee II—"Considerations in the Selection of Slab Dimensions," November, 1956, JOURNAL; Subcommittee III—"Structural Design Considerations for Pavement Joints," July, 1956, JOURNAL; and Subcommittee IV—"Design Considerations for Concrete Pavement Reinforcement for Crack Control," October, 1956, JOURNAL.

The committee sponsored a session at the 54th annual convention in 1958 featuring the reports of three subcommittees on prestressed concrete pavements; use of continuous reinforcement in highway pavements; and design of concrete overlays for concrete pavements. These reports appeared in ACI *Proceedings* V. 55: Subcommittee VI—"Prestressed Pavement—A World View of its Status," February, 1959, JOURNAL; Subcommittee VII—"Continuous Reinforcement in Highway Pavements," December, 1958, JOURNAL; and Subcommittee VIII—"Design of Concrete Overlays for Pavements," September, 1958, JOURNAL.

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Committee 326—Shear and Diagonal Tension—Joint ACI-ASCE

Organized 1950

Mission—A joint committee of ACI and ASCE was formed in 1950 with the assignment of developing methods for designing reinforced concrete members to resist shear and diagonal tension consistent with the new ultimate strength design methods.

Activities—An initial test program investigated shearing strength of a reinforced concrete slab under a centrally located concentrated load. A program of tests on the shear strength of concrete members without shear reinforcement, sponsored by the Reinforced Concrete Research Council, was recently completed at the University of Illinois.

The Committee on Reinforced Concrete Research of the American Iron and Steel Institute is also sponsoring an investigation at Columbia University on the diagonal tension resistance of reinforced concrete beams which are subjected to multiple loads simulating a uniformly distributed load.

Plans by the committee include an investigation of knee-frames to be performed of the University of Illinois also under the sponsorship of the Committee on Reinforced Concrete Research.

Progress reports of committee work were presented at the 55th annual convention, February, 1959, and the 56th annual convention, March, 1960.

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Committee 327—Ultimate Load Design—Joint ACI-ASCE

Organized 1952
Discharged 1957

A joint committee was formed in 1952 with the Subcommittee on Ultimate Load Design of the ASCE Committee on Masonry and Reinforced Concrete, Structural Division. The assignment was to evaluate and correlate theories and data bearing on ultimate load design procedures with a view to establishing them as accepted practice.

A report was issued in 1955 and published by both sponsoring organizations: "Ultimate Strength Design," Paper 809, ASCE *Proceedings*, V. 81, Oct. 1955; ACI JOURNAL, January, 1956.

Portions of this report were abstracted to form the appendix to the 1956 ACI Building Code, which recognizes ultimate strength for the design of reinforced concrete members.

Committee 328—Limit Design—Joint ACI-ASCE

Organized 1956

Mission—The mission of this committee is to develop a recommended practice for the application of limit analysis to reinforced concrete structures. Its work will be an extension of that of ACI-ASCE Committee 327, Ultimate Load Design.

Activities—A comprehensive review has been prepared within the committee which will form a nucleus for subsequent theoretical and experimental studies by the joint committee. Theoretical studies on the amount of redistribution occurring with various percentages of reinforcement have been started. These theoretical studies will be used as a guide for determining the amount of experimentation required.

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Committee 331—Structures of Concrete Masonry Units

Organized 1953

Mission—Committee 331 has been assigned the mission to develop methods of structural analysis and design recommend design and construction practices, to provide users of concrete block, brick, and tile with authoritative information and guidance.

Activities—Subcommittee assignments are (A) methods of structural analysis and design and recommended design practices, (B) recommended construction practices, (C) control of wall movements, and (D) nonstructural properties.

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Texas Industries, Inc.

Committee 332—Recommended Practice for Residential Concrete Work

Organized 1953

Mission—Committee 332's initial mission was to prepare a recommended practice in the field of residential concrete work.

Activities—The committee is already actively engaged in studying the early publication of guide details on recommended good practice on residential work, with the first project, concrete floor slabs on grade, nearly completed. Subsequent committee work will take up types of simplified concrete foundations.

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John T. Young
Central Arizona Concrete Association

Committee 333—Design and Construction of Composite Structures—Joint ACI-ASCE

Organized 1956

Mission—This committee is to study the integration of structural members of dissimilar materials in bridges and buildings. The objective is to prepare recommendations for the design and construction of structures composed of prefabricated beams combined with cast-in-place slabs.

Activities—Composite combinations using concrete include two subjects already under study. One is a research proposal on the connection between precast concrete beams and a cast-in-place slab; the other is a literature survey or bibliography on the bond between a steel beam and a concrete slab.

The committee intends to embark on a systematic study of composite construction. It is expected that such a study will consist of the review of several specific problems, e.g., working load design, ultimate load design, volume changes, shear connections, with the aim of collecting and correlating existing information and pointing out areas where research is needed. The next step would be to outline and execute research projects; and finally, the possible preparation of a manual for composite design and construction.

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University of Houston

Committee 334—Concrete Shell Structures
Organized 1957

Mission—The committee is to review present knowledge in the entire field of shell structures, including both theory and applications and to prepare an initial report on present practice in analysis, design, and specifications for materials and construction. Ultimately, the committee is to report recommended practice for design and specifications to supplement the provisions in the ACI Building Code as modifications necessary to suit the special conditions of shell structures.

The committee will consider all types of concrete shell structures and all materials and methods of construction of which applications are available for study. Design recommendations for cylindrical shell and folded plate roofs may be made by reference to reports of other organizations investigating these types. Recommendations for shell-type storage structures may be made by reference to reports of ACI Committee 714.

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Committee 335—Deflection of Concrete Building Structures
Organized 1957

Mission—Committee 335 will study available research on deflection of concrete flexural members in building structures under rapid and long-term loads to develop recommendations for the prediction of such deflection. Suggested limits for allowable deflection of various types of concrete flexural members under various classes of loadings will be developed. The recommendations for prediction and limits of deflection shall be correlated with limitations on acceptable deflections, minimum depth requirements, and allowable limits of deflection and recovery under test loads prescribed in existing ACI Standards. First report is to cover existing satisfactory practice for ordinary structures and eventually to include all structures.

Activities—Field and laboratory research results will be studied. The studies will include effects of aggregate (lightweight or normal); high strength concretes; precasting-plant curing procedures; age and drying; continuity; amount, type, and location of reinforcement; prestressing; and composite action of precast with cast-in-place construction. Types of members will include beams, two-way and flat slabs, arches, folded plate and thin shell structures, and rigid frames. Classes of loading will include uniform and concentrated; wind, blast, or earthquake; moving, temporary; and permanent. Reference to and correlation with the reports of other committees will be expected.

This proposed scope is limited to building structures excluding bridges, mass concrete structures, tunnels, and storage structures. It could be further

restricted to common building elements if desired, excluding arches, folded plates, and thin shells.

M. V. Pregnoff, Chairman
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Stanford University
Carlos D. Bullock
Burgwin and Martin
Adrian E. Eckberg
Charles T. Main, Inc.†
Charles M. Herd*
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John K. Minasian
Structural Engineer
Douglas E. Parsons
National Bureau of Standards
Donald R. Pierce
Ammann and Whitney
A. C. Scordelis
University of California
Mete Sozen
University of Illinois

Committee 336—Combined Footings
Organized 1958

Mission—Committee will collect records of successful designs, compare theoretical assumptions of soil behavior, and compare design methods in use and/or those prescribed by other codes. The aim of this committee is to prepare design recommendations for safe standard practices covering all types of foundations except isolated footings.

Activities—All types of combined footings now outside the scope of the ACI Building Code will be considered. Generally accepted methods of design presented in various reference books, authorized by various statutory codes, and those in use by practicing design offices will be considered. In particular, performance records will be correlated with design methods in developing recommendations.

Jacob Feld, Chairman
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E. I. Fiesenheiser
Illinois Institute of Technology
W. Fuchssteiner
Consulting Engineer, Germany
F. D. C. Henry*
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Albert Kahn Associated Architects and Engineers, Inc.†
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Civil Engineers
Andrew Reti
Civil Engineer
Paul Rogers
Paul Rogers and Associates
J. George Thon
Bechtel Corp.†
Ebenezer Vey*
Illinois Institute of Technology
Max Zar
Sargent and Lundy

*Nonmember ACI.
†Corporation Member.

Committee 337—Strength Evaluation of Existing Concrete Structures
Organized 1958

Mission—Committee will collect information on methods of evaluating capacity of existing concrete structures. Reports will include recommendations for a theoretical analysis, field data required, and methods of securing such data.

Activities—Now in the process of organizing, the committee will review available information on the analysis and evaluation of the effects of fire, weathering, abrasion, chemical attack, settlement, cracks, frozen or faulty joints, openings cut, and additional work added after completion of construction. Recommendations will be made relative to the evaluation of strength, pulse velocity, surface hardness, abrasion resistance, sonic modulus, permeability, and creep.

Carlos D. Bullock
Burgwin and Martin
James S. Blackman
University of Nebraska
Keith H. Driggs
Utah Road Commission
Jacob Feld
Consulting Engineer
James R. Libby
San Diego Prestressed Concrete
Ernst Maag
California Division of Architecture
A. S. Neiman
Engineering Division, U. S. Air Force
F. W. Panhorst
California Division of Highways
Adrian Pauw
University of Missouri
Abraham Pikoos
Wilmington, Delaware
Herbert M. Stoll
A. Epstein & Sons
Charles W. Yoder
Charles W. Yoder and Associates

Committee 338—Torsion
Organized 1958

Mission—Committee will study available research, both theoretical and lab-

Expressway, Elmsford Section, Westchester County, N. Y., Contractor: Mount Vernon Contr. Corp., Chief Engineer: Philip P. Fox, Designer: Charles H. Sells, Consulting Engineer of New York City. Photo: Fred & Leo de Wys

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oratory, and existing requirements for design and statutory codes. The report will include recommendations for allowable torsional stresses, singly and in combination with shear, flexural, or axial stresses for plain and reinforced concrete. Recommended methods for analysis and design for elastic or ultimate strength methods will be given.

Activities—The committee will cover design of plain, reinforced, or prestressed concrete for torsion, special reinforcement for torsion; and combinations of spiral, hoop, and tie reinforcement with longitudinal reinforcement. Studies of research will probably include results of elastic and plastic theory compared to strength tests.

GORDON P. FISHER, Chairman
Cornell University
DAVID EHRENPREIS
Consulting Engineer
RUDOLPH SZILARD
The Martin Co.
PAUL ZIA
University of Florida

Committee 339—Allowable Stresses in Reinforcement

Organized 1958

Mission — Committee will analyze available published research data, foreign practice, and available examples in this country utilizing high strength reinforcement in nonprestressed concrete. The initial report may be limited to a summary of successful practice in which allowable steel stress is related to strength of reinforcement including limitations which were imposed on, and prescribed methods of estimating, crack width and deflection. Necessary liaison with reinforcing steel producers and the Comité Européen du Béton should be established.

Activities—The committee will limit itself to evaluating successful practice and research rather than initiating research. The objectives of its report will be to present an evaluation of performance and a comparison of safety factors in practice rather than to develop recommendations for safety factors. A progress report was given at the 56th annual convention, March, 1960.

ROBERT SAILER, Chairman
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Ketchum, Konkel and Hastings
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Tor-Isteg Steel Corporation, Austria†
GEORG WASTLUND
Cement and Concrete Institute, Sweden
DAVID WATSTEIN
National Bureau of Standards
A. CARL WEBER
Laclede Steel Company‡
GEORGE WINTER
Cornell University

Committee 340—Ultimate Strength Design Handbook

Organized 1958

Mission—The initial mission is to develop a handbook containing data and design aids for reinforced concrete design by the ultimate strength method as authorized in the "Building Code Requirements for Reinforced Concrete (ACI 318-56)." The standing mission will be to maintain the handbook in accordance with requirements of future revisions to the ACI Building Code.

Activities—The committee will produce a handbook which will be published as a separate publication of the Institute. The committee will use available material as much as possible. The handbook may contain papers or portions thereof which were previously published in the ACI JOURNAL. Under the initial mission, emphasis will be on USD for flexure and/or axial load. Aids to ultimate design for shear, bond, and deflection may be included.

EDWARD COHEN, Chairman
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Bureau of Reclamation
J. J. O'SULLIVAN
The Rand Corp.
ALFRED L. PARME
Portland Cement Association
GEORGE WINTER
Cornell University

*Nonmember ACI.
†Corporation Member.
‡Contributing Member.

Committee 401—Specifications for Structural Concrete
Organized 1956

Mission—The committee is to prepare material to serve as a basis for the section on structural concrete in specifications. The report is to be in such form that applicable portions may be incorporated and additional special provisions inserted as required by the specification writer to form a complete specification for structural concrete. When generally accepted requirements in legal phraseology are available, i.e., appropriate sections of other ACI Standards or ASTM specifications, the provisions will be merely references. Other provisions may incorporate excerpts from generally accepted ACI reports and papers, including the Joint Committee Report of 1940, or recommendations based upon such generally accepted material may be prepared in legal phrasing by the committee. Suggested title for report is "Guide to Specifications for Structural Concrete."

Activities—The committee work has been divided among 12 subcommittees. Many of the subcommittees have prepared initial drafts of sections of the report. Progress report of committee work was presented at the 56th annual convention, March, 1960.

GEORGE H. NELSON
Law Engineering Testing Co.
JULIUS ADLER*
Consulting Engineer
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H. B. BRITTON
New York State Department of Public Works
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Consulting Engineer

*Nonmember ACI.
†Corporation Member.
‡Contributing Member.

HERBERT M. SHILESTONE, JR.
Shilestone Testing Laboratory
JOHN J. WHITE
Parsons, Brinkerhoff, Hall and MacDonald

Committee 402—Concrete Floor Finishes
Organized 1957

Mission—The committee will review all earlier reports of ACI committees and papers by individuals on concrete floor finishes, suggest a standard abrasion test, and prepare a recommended practice on materials and construction methods. The initial report of the committee will suggest a standard abrasion test and recommend research needed to establish a scale of performance ratings suitable for specifications. The final report of the committee will include a specification on concrete floor finishes supplementing the work of Committee 401 — Specifications for Structural Concrete.

The entire range of concrete floor finishes including heavy-duty types for exposure to industrial traffic wear and aggressive chemicals and exposed decorative floors will be studied. General classes of proprietary materials including metallic, carborundum, and other special aggregates and liquid chemical hardeners will be considered. A scale of performance ratings based on a standard wear test will be established and recommended ranges of ratings for various classes of use established.

HOWARD J. MCGINNIS
Portland Cement Association
A. T. HERSEY
Alpha Portland Cement Co.‡
GERALD MILSOM
Metallicrete Floor Co., Ltd., Montreal

Committee 505—Design and Construction of Reinforced Concrete Chimneys
Organized 1922

Mission—This committee was reorganized in 1959 to restudy the present standard with a view to its suitability in the light of new developments and to recommend any modifications necessary to satisfy present or anticipated operating conditions in reinforced concrete chimneys. The existing standard, "Specification for the Design and Construction of Reinforced Concrete Chim-

neys (ACI 505-54)," was adopted at the 50th annual convention, ratified by letter ballot in June, 1954, and published in the September, 1954, ACI JOURNAL.

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Commonwealth Edison Co.

Committee 604—Winter Concreting Methods
Organized 1929
Discharged 1957

A revised standard was adopted at the 1956 convention and subsequently ratified by letter ballot. The new standard "Recommended Practice for Winter Concreting (ACI 604-56)," was published in the June, 1956, ACI JOURNAL; it superseded ACI 604-48.

Committee 605—Hot Weather Concreting
Organized 1955
Discharged 1960

The committee was organized to review information on the effect of hot weather on the characteristics of fresh and hardened concrete, and to develop a recommended practice for controlling ingredients, batching, mixing, transporting, placing, protection, and curing concrete during hot weather.

A progress report, "Hot Weather Concreting Problems," was presented at the 1957 annual convention and published in the May, 1957, JOURNAL (*Proceedings* V. 53).

The committee report, "Recommended Practice for Hot Weather Concreting (ACI 605-59)," was adopted at the 55th annual convention and ratified by letter ballot in June, 1959. The proposed standard appeared in the November, 1958, JOURNAL. The standard, adopted without change, has been printed separately by the Institute.

Committee 609—Consolidation of Concrete
Organized 1934

Mission—This committee has the assignment to revise and bring up to date a former report (ACI JOURNAL, Mar.-Apr. 1936, *Proceedings* V. 32), and has been organized to encourage research to discover the basic effects within concrete produced by vibrators of different design, and to determine how their characteristics can be used to the greatest advantage.

Activities—A symposium on vibration practice was presented by the committee at the 1953 convention. In 1959, the title of the committee was changed from "Compaction of Concrete by Mechanical Means."

The committee report "Consolidation of Concrete" appeared in the April, 1960, JOURNAL. A progress report of committee work was presented at the 56th annual convention, March, 1960.

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Knoerle, Graef, Bender and Associates
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University of Wisconsin
GEORGE WERNER
Bureau of Public Roads

Committee 611—Inspection of Concrete
Organized 1935

Mission—The standing mission of this committee is to develop the *ACI Manual of Concrete Inspection* and keep it current by periodic revision. The mission also includes the promotion of effective inspection in every possible

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way through such means as encouraging greater recognition of inspectors and inspection, publication of reports on effective inspection, and encouragement of short courses and conferences on inspection.

Activities—The fourth edition of the manual was published in 1957. The committee has also fostered publication of papers on aspects of inspection in the ACI JOURNAL, *Western Construction*, and AIA *Bulletin;* other papers are in preparation.

M. R. SMITH, Chairman
Office, Chief of Engineers, Department of the Army
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California State Department of Water Resources
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Portland Cement Association
JOSEPH J. WADDELL
Knoerle, Graef, Bender and Associates
D. O. WOOLF
Bureau of Public Roads

Committee 612—Recommended Practice for Curing Concrete

Organized 1936

Mission—This committee is to prepare a recommended practice for curing concrete.

Activities — The committee report "Curing Concrete" was published in the August, 1958, JOURNAL. A proposed standard containing recommendations in specification form is now under consideration. The committee has sponsored basic research at the National Bureau of Standards on curing methods, and a symposium on curing was sponsored at the 1952 convention.

EDWARD E. BAUER, Chairman
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Bureau of Reclamation
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Caterpillar Tractor Co.
W. R. JOHNSON
Khuzestan Development Service, Iran
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Waterways Experiment Station
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Huron Portland Cement Co.‡
MARK MORRIS
Iowa State Highways Commission
F. V. REAGEL
Missouri State Highway Commission†
GEORGE WERNER
Bureau of Public Roads

Committee 613—Recommended Practice for Proportioning Concrete Mixes

Organized 1936

Mission—This committee has the assignment of developing recommended practices for proportioning concrete mixes, including air-entrained and non-air-entrained concrete, lightweight concrete, and no-slump concrete.

Activities—The committee has published the standard, "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," (ACI JOURNAL, September, 1954), which superseded ACI 613-44.

"Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 613A-59)," a subcommittee report which supplements ACI 613-54, was approved as an ACI Standard at the 55th annual convention and ratified by letter ballot in June, 1959. The proposed standard was published in the September, 1958, JOURNAL. The standard, adopted without change, has been printed separately by the Institute. Another subcommittee is actively studying the problems involved in preparing recommendations for proportioning mixes for no-slump concrete, with particular reference to block, pipe, and other precast products.

CLAYTON L. DAVIS, Chairman
Universal Atlas Cement Division, U. S. Steel Corp.‡
ROBERT A. BURMEISTER
City of Milwaukee
W. A. CORDON
Utah State University†
ROBERT K. DUEY
Consulting Engineer

†Corporation Member.
‡Contributing Member.

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Expanded Shale, Clay and Slate Institute†
RICHARD J. FRAZIER
Harry T. Campbell Sons' Corp.
A. T. HERSEY
Alpha Portland Cement Co.‡
FRED HUBBARD
National Slag Association
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Consulting Engineer
THOMAS B. KENNEDY
Waterways Experiment Station
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Portland Cement Association
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Wisconsin State Highway Commission
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Bureau of Reclamation
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Hydro-Electric Power Commission of Ontario†
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Herculite Light Weight Aggregates of Canada, Ltd.
I. L. TYLER
Portland Cement Association
STANTON WALKER
National Sand and Gravel Association‡ and National Ready Mixed Concrete Association‡
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BENJAMIN WILK
Standard Building Products Co.
CEDRIC WILLSON
Texas Industries, Inc.
PAUL M. WOODWORTH
Waylite Co.
JOHN C. WYCOFF
Solite Corp.†

Committee 614—Recommended Practice in Measuring, Mixing, and Placing Concrete
Organized 1936
Discharged 1960

The committee, discharged after completion of the 1942 standard, was reorganized in 1956. A revised standard, "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)," was adopted at the 55th annual convention and ratified by letter ballot in June, 1959, to supersede ACI 614-42. The proposed standard was published in the November, 1958, JOURNAL with errata published in the July, 1959, JOURNAL. The standard has been printed separately by the Institute.

*Nonmember ACI.
†Corporation Member.
‡Contributing Member.

Committee 616—Coatings for Concrete
Organized 1936

Mission—The committee's assignment is to prepare recommended practices for the application of paint and coatings of different kinds to concrete surfaces, including industrial applications of coatings to prevent deterioration of concrete, damproofing, and waterproofing.

Activities—The committee prepared the standard "Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)," September, 1949, JOURNAL. The report "Guide for Painting Concrete," for paints other than portland cement paint, was published in the March, 1957, JOURNAL (*Proceedings* V. 53).

A start has been made on preparation of a bibliography on the painting of concrete. Future assignments are: (1) industrial applications of coatings to prevent chemical attack on concrete, and (2) dampproofing and waterproofing of concrete.

WILLIAM H. KUENNING, Chairman
Portland Cement Association
G. E. BURNETT
Bureau of Reclamation
JOHN C. MOORE*
Coatings Research Group, Inc.
JOHN L. ROHWEDDER
Rock Island District, Corps of Engineers
BRUCE R. SCHOENFELD
Consulting Engineer
FRANCIS SCOFIELD*
National Paint, Varnish and Lacquer Association
C. A. SENTAL*
National Bureau of Standards
KENNETH TATOR*
Kenneth Tator Associates
D. O. WOOLF
Bureau of Public Roads

Committee 617—Specifications and Recommended Practice for Concrete Pavements and Bases
Organized 1937
Discharged 1959

The standard "Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)," superseding ACI 617-51, was adopted at the 54th annual convention, ratified by letter ballot, and published in the July, 1958, JOURNAL. The specification complements the recommendations for design by Committee 325.

Another New York skyscraper has been reinforced with USS American Welded Wire Fabric. This is Rockefeller Center's new forty-eight story Time & Life Building, an outstanding example of contemporary architectural design. The exterior steel columns are encased in stone-faced concrete which project from the walls and serve to accent the vertical sweep of the tower.

The frame supports short span, lightweight concrete slabs reinforced with USS American Welded Wire Fabric. Each slab is 8'0" long and 4" thick. When asked why the fabric-reinforced short-span design was selected for this structure, W. B. Scofield, partner in the structural engineering firm of Edwards & Hjorth, said "This system provides first-class, fireproof construction with a long record of satisfactory service in addition to its proven economy, speed of construction, and occupancy flexibility."

USS American Welded Wire Fabric was also used to reinforce the concrete fireproofing encasement of the columns, girders, and beams. Fabric is excellent for this application because the small, closely spaced members reinforce this thin concrete best. In addition, fabric is easily shaped to fit the contours and is sufficiently rigid to maintain the required shape.

Please write American Steel & Wire, Dept. 0250, 614 Superior Avenue, N.W., Cleveland, Ohio or contact our nearest sales office for complete information on these or any other uses of USS American Welded Wire Fabric.

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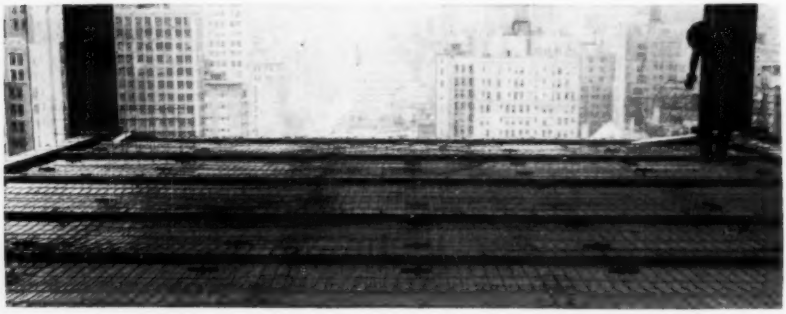
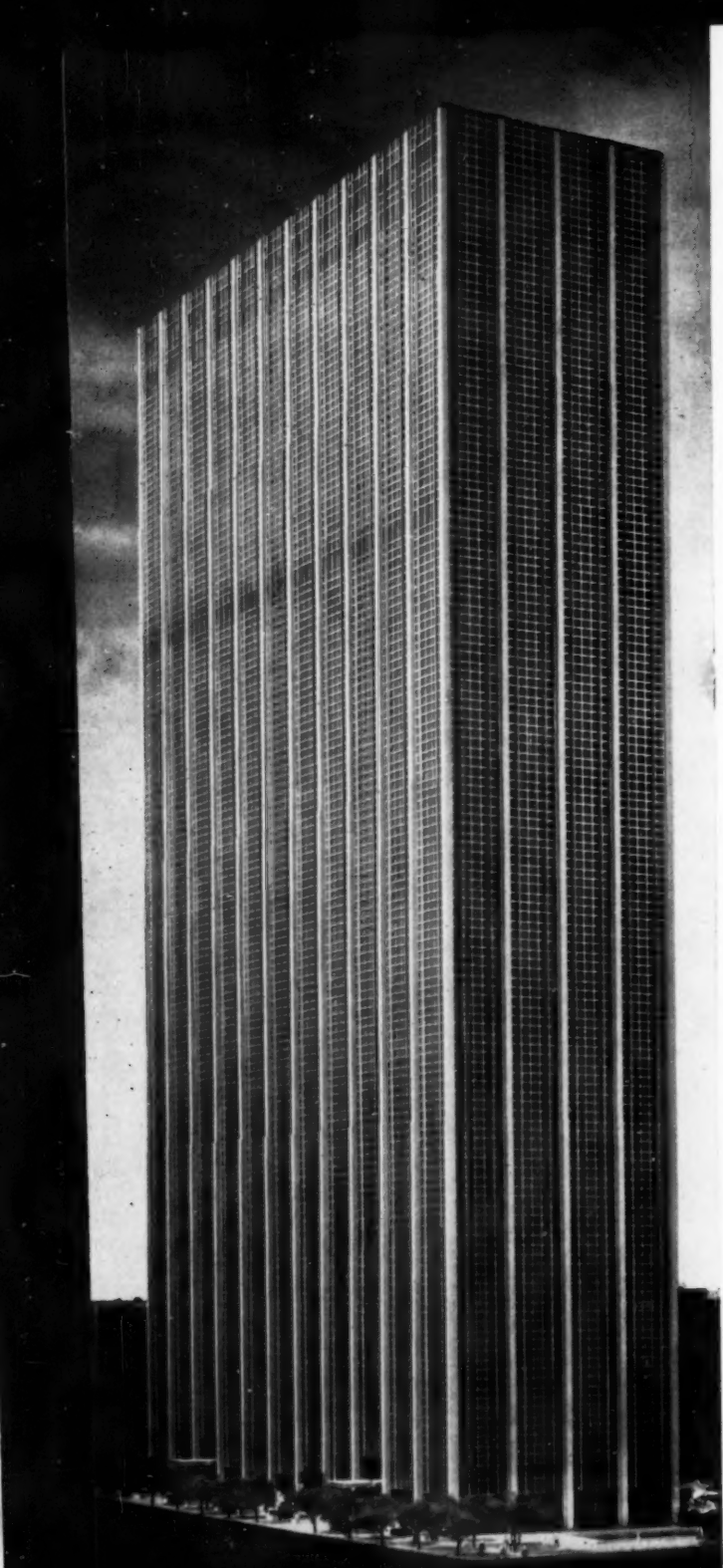


Short-span fabric-reinforced floor system in Rockefeller Center's Time & Life Building. American Welded Wire Fabric was furnished in long rolls and merely unrolled perpendicular to the beams and on top of the forms. It was draped from the top of the slab over the beams to the bottom of the slab at mid-span. Thus, the reinforcement is in position to best resist both positive and negative moments. The economy of steel placement is apparent. In total, over six million square feet of short span slabs reinforced with Welded Wire Fabric have been used in New York's Rockefeller Center.

Owners: Rockefeller Center, Inc. and Time Inc.
Architects: Harrison & Abramovitz & Harris
Structural Engineers: Edwards & Hjorth
General Contractor: George A. Fuller Company, John Lowry, Inc.
Fabric Distributor: Fireproof Products, Inc.



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Columbia-Geneva Steel Division,
San Francisco, Pacific Coast Distributors
Tennessee Coal & Iron Division,
Fairfield, Alabama, Southern Distributors
United States Steel Export Company,
Distributors Abroad

Committee 621—Aggregates
Organized 1936

Mission—This committee was reorganized in 1955 with the mission of preparing a statement of information on aggregate that will evaluate properties of aggregate in terms of their influence on properties of concrete, discuss features of preparation and handling which have a bearing on concrete quality and uniformity, and in general, summarize what should concern the user about aggregate when he sets forth to do a first-class job.

Activities — Previous work of the committee yielded papers by committee members on production of sand and gravel, crushed stone, and slag. A bibliography on mineral aggregates for concrete will be submitted to the Technical Activities Committee shortly.

WILLIAM R. WAUGH, Chairman
Office, Chief of Engineers, Department of the Army
ROBERT E. PHILLEO, Secretary
Office, Chief of Engineers, Department of the Army
E. W. BAUMAN
National Slag Association
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Hydro-Electric Power Commission of Ontario†
C. E. GOLSON
Western Machinery Co.
JOSEPH E. GRAY
National Crushed Stone Association†
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Southern California Edison Co.
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California State Department of Water Resources
I. L. TYLER
Portland Cement Association
STANTON WALKER
National Sand and Gravel Association‡ and National Ready Mixed Concrete Association‡
D. O. WOOLF
Bureau of Public Roads

Committee 622—Formwork for Concrete
Organized 1955

Mission—The committee's assignment is to recommend practice for concrete formwork in general terms, based on considerations of safety and appearance. Consideration is directed toward: (1) formulas for estimating maximum form pressure; (2) allowable stresses on form, shore, and scaffolding materials; accessories; soil; bracing and guys; etc., to be expressed as a fraction of the ultimate strength of the material; (3) allowable limits on deflection and/or strains in the above items; (4) allowable tolerances in general; (5) special limits for items (2), (3), (4) applicable to architectural concrete; (6) a suggested specification for use by architects and engineers; (7) special requirements for accessories to be used on architectural concrete; (8) forming at joints; (9) section dealing with special cases such as grouting in aggregate-filled forms, forming cast-in-place material composite with precast.

Activities—Subcommittee II's report, "Form Construction Practices," was published in the June, 1957, JOURNAL. A preliminary report, "Pressures on Formwork," was published in the August, 1958, JOURNAL. Work on a recommended practice for design and construction of concrete formwork is well advanced.

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Committee 623—Cellular Concretes
Organized 1958

Mission—The committee will prepare recommendations for the use of cellular concretes. Specifications for materials and construction upon which such recommendations are based will be included. If necessary, special tests for compliance with the specifications may be suggested either for samples or for in-place construction.

A secondary standing assignment will be to furnish information on special properties of cellular concretes for use by Committee 213.

Activities—In 1959, the committee title was changed from "Specifications and Practices for Foamed Concrete." The committee will review research data including the report "Cellular Concrete," ACI JOURNAL, May and June, 1954, *Proceedings* V. 50, and later research results. Data on performance, construction practice, and design practice will be collected and evaluated. Tests for control and acceptance in use will be considered.

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Committee 624—Portland Cement Plaster
Organized 1958

Mission—Committee assignment is to prepare a specification for portland cement plastering.

Activities—The committee will study the recommendations of the former committee, other recommendations, research, and present practices. Recommendations will be developed for interior and exterior application, reinforcement or backing required, dec-

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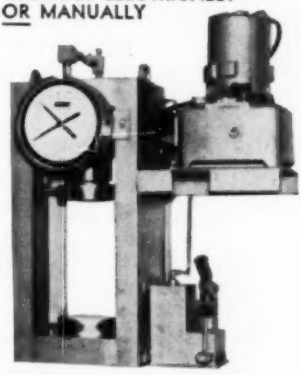

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Committee 704—Cast Stone
Organized 1928
Discharged 1949

The report of this committee, "Specification for Cast Stone (ACI 704-44)," was published in the June, 1945, ACI JOURNAL. The report was dropped as an Institute standard in 1956 because industry interest was insufficient to reactivate the committee to restudy the subject and keep the report current.

Committee 711—Precast Floor Systems for Houses
Organized 1934
Discharged 1959

The report of this committee, "Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)," superseding ACI 711-53, was adopted at the 54th annual convention, ratified by letter ballot, and published in the July, 1958, JOURNAL.

Committee 712—Precast Structural Concrete Design and Construction—Joint ACI-ASCE
Organized 1957

Mission—The mission of the committee is to develop a recommended practice to cover the over-all problems of design and assembly of beams, slabs, columns, and other precast concrete members. Under consideration is the inclusion of precast prestressed members (particularly pretensioned), lift-slab, and tilt-up construction.

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Committee 714—Concrete Bins and Silos
Organized 1937

Mission—This committee is to prepare recommendations for the design and construction of concrete structures for the storage of solids. The report is to include data on static design pressure for common solids, such as cement, flour, sugar, grain, and coal. Design recommendations will cover use of prestressed concrete and will include folded-plate analysis for circular and prismatic structures, allowance for dynamic pressure in loading and unloading, foundations to resist overturning from lateral or unbalanced loads, structural details to prevent local failure, multi-cellular and single units, and any modifications of the ACI Building Code necessary to suit special conditions in such storage structures. Recommendations for specifications will include any necessary provisions peculiar to such storage structures and any special protective treatments required for storage of particular common bulk materials. The recommendations will include any special precautions, limitations, and

*Nonmember ACI.
†Corporation Member.

requirements for the sliding-form or shotcrete methods of construction.

Activities—With its reactivation in 1956, the committee's title and mission were changed to that above. The original committee, discharged in 1948, prepared the ACI Standard "Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)," published in the October, 1946, ACI JOURNAL.

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Portland Cement Association

Committee 716—High Pressure Steam Curing
Organized 1941

Mission—The committee was assigned the task of studying, correlating information, and reporting on the subject of high-pressure steam curing.

Activities—Reactivated in 1950 because of an awakening of interest in high-pressure steam curing, the new group began a study of the report of former Committee 716 (ACI JOURNAL, April, 1944, *Proceedings* V. 40, p. 409) considering possible additions to the old report.

Committee work has resulted in reports published in the April and May, 1953, issues of the JOURNAL ("Physical Properties of High-Pressure Steam-Cured Concrete Block" and "Chemical Reactions in High-Pressure Steam Curing of Portland Cement Products," respectively).

In 1958, a series of shrinkage tests were begun, sponsored jointly with Committee 213. Four laboratories are participating: the National Bureau of Standards, National Research Council of Canada, Portland Cement Association, and Waterways Experiment Station.

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Solite Corp.†

Committee 717—Practice in Low-Pressure Steam Curing
Organized 1958

Mission—Committee will prepare a comprehensive guide to the best practice for using low pressure steam and high temperatures in curing concrete. Recommended practices will be prepared for accelerated curing of structural concrete elements and common types of precast concrete products.

*Nonmember ACI.
†Corporation Member.
‡Contributing Member.

Recommendations will be used on present practice and presently available research data. Research data collected and recommendations developed by other ACI committees will be considered for incorporation in the report.

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East Bay Municipal Utility District†

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Committee 805—Application of Mortar by Pneumatic Pressure
Organized 1942

Mission—Committee will review the existing standard "Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 850-51)" [ACI JOURNAL, May, 1951, *Proceedings* V. 47] and prepare a revision. Objectives of the revision will be to reflect new developments in this field since 1951 extending the scope of the original report as necessary and to expand sections presently requiring clarification.

Activities—The committee, reactivated in 1957, will extend the scope of the report to include limits of applicability

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and recommendations for use of plastic mix shotcrete, for admixtures suitable in shotcrete, for lightweight aggregate shotcrete, for use of cements other than portland, and for construction of thick structural members.

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Corps of Engineers

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San Francisco Port Authority

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Joint Committee
Organized 1904
Discharged 1942

The first Joint Committee was formed in 1904 at the beginning of reinforced concrete construction in the United States. A second was formed in 1920 and a third in 1930. The report of the last Joint Committee, "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," was published in 1940. Represented on the committee were ACI, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, and Portland Cement Association.

A joint ACI-ASCE steering committee was formed in 1953 to study the 1940 report with a view to future action on it. The committee recommendation that the 1940 Joint Committee Report be discontinued was approved by both organizations in 1956, and all affiliated groups were notified.

ACI Participation in the Work of Committees of Other Organizations

ASA Sectional Committee (A1) Specifications and Methods of Test for Hydraulic Cements

MILES N. CLAIR
Thompson and Lichtner Co., Inc.†

ASA Sectional Committee (A37) Road and Paving Materials

HAROLD ALLEN
Bureau of Public Roads

ASA Sectional Committee (A41) Masonry

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Lehigh Portland Cement Co.‡

ASA Sectional Committee (A42) Plastering

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ASA Sectional Committee (Z23) Sieves

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Office, Chief of Engineers,
Department of the Army

ASTM Committee (C1) Cement

THOMAS B. KENNEDY
Waterways Experiment Station

ASTM Committee (C9) Concrete and Concrete Aggregates

KENNETH D. CUMMINS
American Concrete Institute

Subcommittee I-d, Editorial and Definitions, ASTM Committee (C9) Concrete and Concrete Aggregates

EDWARD E. BAUER
University of Illinois

ASTM Committee (E5) Fire Tests of Materials and Construction

G. E. TROXELL
University of California

American Welding Society Committee on Welding Concrete Reinforcing Steel

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Graphic Arts Industry, Inc.—Task Group on Building Facilities, Committee on Engineering and Materials Handling, Research and Engineering Council

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International Federation of Prestressed Concrete

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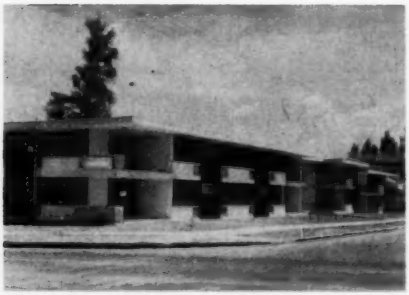
Reinforced Concrete Research Council

RAYMOND C. REESE
Consulting Engineer

Standing Committee for Simplified Practice Recommendation R163-48, Coarse Aggregates (Crushed Stone, Gravel and Slag). Commodity Standards Division, U. S. Department of Commerce

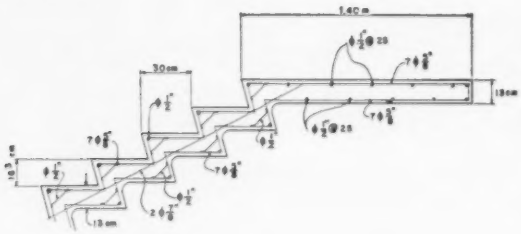
F. V. REAGEL
Missouri State Highway Department†

†Corporation Member.



Interesting Concrete Stairway

An apartment building completed recently in Barranquilla, Colombia, features some interesting reinforced concrete stairways. The accordion type stairway has no support at the intermediate landing. The stairway is 5 in. thick and is free from the columns and walls, being supported only at the base footing and the top landing. The stairway was designed for a live load of 300 kg per sq m (60 psf).



The stairway was load tested with a 2000-kg load distributed on the landing (700 kg) and on the steps. A 2-m deflection was observed at the edge of the landing. When the stairway was unloaded, the deflection disappeared.

The apartment building is owned by Mrs. Ernestina Santodomingo and was built by the engineering and contracting firm of Cornelissen and Compania Ltda., Barranquilla, Colombia. The stairway was design by Luis Paredes Manrique.

Four Digesters and Six Settling Tanks Constructed of Prestressed Concrete

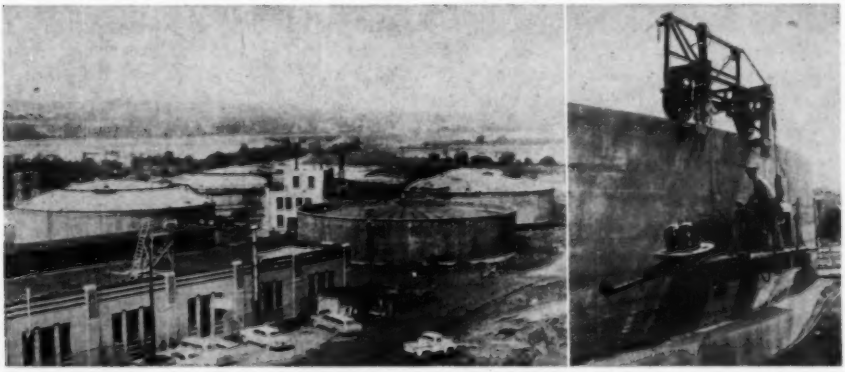
FOUR DIGESTERS and six settling tanks, all of prestressed concrete construction, are now being erected at the Metropolitan Syracuse Treatment Plant of the Onondaga Public Works Commission, Onondaga County, N.Y. The digesters measure 100 ft in diameter by 30 ft 4 in. high and the settling tanks are 122 ft in diameter by 13 ft 1 in. high. The digesters will more or less float in the muck prevalent in the valley area and were designed to withstand especially large stresses resulting from large settlements.

Digester concrete base slabs are 3 ft thick while the settling tank slabs vary from 5 ft at the center to 2 ft 6 in. at the edge. The walls of the digesters are of 8 in. thick cast-in-place concrete whereas those of the settling tanks are of 5 in. thick pneumatically applied mortar. The settling tanks are open vessels but the digesters are covered.

The roofs of three digesters are of concrete and one has a floating steel cover. The concrete roofs are dome shaped, 10 in. thick at the edges, and taper to a 2½ in thickness at the center. Eight in. thick lean concrete covers were placed over the structural roofs to counteract the stress set up by the gas pressure on the under side of the dome. Prior to applying the covers, bond breakers were applied to prevent the covers from adhering to the structural roofs.

Prestressing wire was applied by a wire-winding machine which travels around the outer surface of the vessel. Dome edges are prestressed with seven wraps of wire whereas the walls require only two. Mortar is pneumatically applied between each layer and the completed job receives a protective mortar covering.

O'Brien & Gere, Syracuse, N. Y., were consulting engineers on the job. The walls of the settling tanks and digesters and the concrete domes of the digesters are being constructed by Preload Concrete Structures, Inc., Westbury, N. Y.



Left—Four prestressed concrete digesters constructed for the Onondaga County Public Works Commission. Six settling tanks are being constructed forward and to the right of the digesters. Right—Wire-winding machine stretches prestressing wire and wraps it around concrete wall

Prefabricated Prestressed "Hypar"

Forms Intricate Roof and Wall Structure

THE PHILIPS PAVILION at the 1958 Brussels World Fair designed by Le Corbusier and Y. Xenakis aroused considerable interest in the world of architecture because of its extraordinary conception and advance design as a shell structure. The building was composed entirely of shells having the form of hyperbolic paraboloids. The method of construction in prestressed concrete was remarkable for its originality and elegance.

Visual, acoustical, and architectural considerations led to the pavilion being built up of hyperbolic paraboloids (hypars).

The membrane theory for the hypar shell gave a general idea of the behavior of the shell structure and allowed solution of some of the detailed problems. However, for a complete calculation of the mechanical properties of such a complex structure as the Philips Pavilion, it was necessary to utilize model tests in the early stages of design and prior to construction.

Tests on a 1:25 scale model in plaster of Paris on a framework of wire gauze with steel tubes simulating the ribs of the structure showed that a structure with 5 cm thick concrete shells and 40 cm thick ribs would be sufficiently strong. In a second model of 1:10 scale, two of the hypar shells were built up from several hundred appropriately shaped plywood slabs. These shells were prestressed and the results proved the feasibility of the proposed system of construction using precast concrete slabs and prestressing wires anchored in the ribs.

Abstracted from a report by H. C. Duyster in *Philips' Technical Review*, V. 20, No. 1, 1958-59.

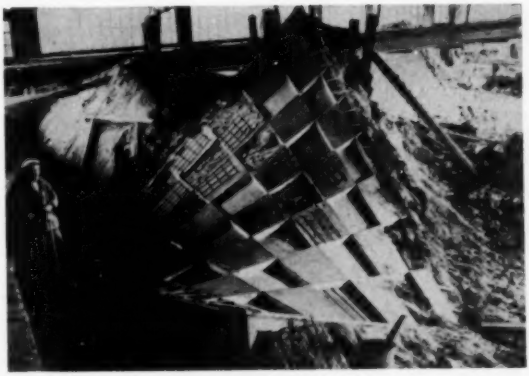


Fig. 1 — Concrete slabs being cast on a sand bed to form a part of a hyperbolic paraboloidal shell. By positioning slats of wood 1 cm thick along the ruling lines (all straight lines) the entire surface of the section is divided into lozenge-shaped molds of approximately 1 sq. m. Each 5 cm thick slab is reinforced with mesh to prevent breakage during handling. The "shell" was cast in checker-board fashion

Why prestressing?

The factors that favored prestressing were:

1. For shells as thin as the ones in the pavilion, the prestressing wires could be applied on the surface. The shape of the walls of the pavilion lent itself well to this treatment due to the fact that hyperbolic paraboloids may be generated by straight lines, thus making it possible to apply all or most of the prestressing wires such that they would run *straight*. To handle the tensile stresses in such a structure with ordinary concrete would have been extremely difficult.

2. By appropriate prestressing of both the shell and the edge members, it is possible to limit the disparities in deformation that are responsible for the occurrence of often relatively large secondary stresses. It then becomes more economical to construct the shell in prestressed concrete than in reinforced concrete for the prestressed shell can take up the load mainly with membrane stresses and hence approaches a true "shell" construction. In a shell of reinforced concrete the secondary stresses produced by deformation will generally predominate and may therefore require thicker walls and possibly additional strengthening with supports.

3. Of practical importance was the fact that less stringent demands were required for the waterproofing of the shells due to the fact that prestressing reduced the likelihood of hairline cracks opening to the extent that there would be leakage. It would have been quite difficult to provide these unusually shaped shells with conventional roofing.

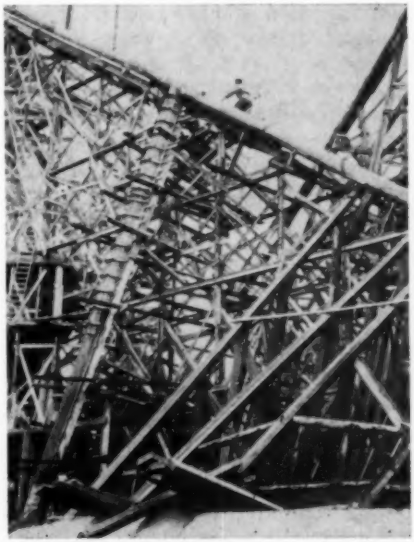


Fig. 2—The 40 cm diameter cylindrical ribs were cast in place

Fig. 3—The precast slabs were placed in position and joints mortared on scaffolding that incorporated wood beams running along the ruling lines of the shells

4. Perhaps one of the most important items for this structure was that prestressing enabled the contractor to form the shell surfaces from precast slabs.

Precasting the shells

It would have been difficult to cast the thin, and in many cases, strongly twisted shells of the pavilion in place on the structure. The contractor therefore devised the plan to form each hypar shell from sections which would be cast in open sand-bed molds.

The sand beds on which the lozenge-shaped concrete slabs were cast—each slab individually molded as part of a particular hyperbolic paraboloid—were made by delineating them, following the ruling lines, with straight planks of wood and then leveling them off (Fig. 1). The mound of sand, containing a certain proportion of loam, was coated with a thin skin of cement after leveling off. By positioning slats of wood 1 cm thick along the ruling lines the entire surface was divided into lozenge-shaped molds of approximately 1 sq m. Each 5 cm thick slab was reinforced with mesh to prevent breakage during transport to the site and during handling.

Erecting the shells

The edge members at the intersections of each pair of hyperbolic paraboloidal shells were cylindrical concrete ribs 40 cm in diameter. They were cast in place (Fig. 2) while the more than 2000 slabs were being cast indoors.

After the ribs had been cast, further scaffolding was erected of which the outer-surface beams again followed the ruling lines of the hyperbolic paraboloidal surfaces. The precast slabs were then placed in position on the scaffolding and held temporarily in place (Fig. 3). The 1 cm wide joints between the slabs were filled with mortar and the entire structure was then prestressed.

Prestressing the shells and ribs

The required prestressing was derived from the model tests. The loads allowed for were the dead weight of the structure together with the sound proofing, a snow load, and a wind load of 75 kg per sq m from various directions.

The necessary prestress was introduced in the shells by systems of 7-mm steel wires laid over the surface and post-tensioned (Fig. 4). Due to lack of time to test models with prestressing applied both on the inside and outside of the shell, it was decided to apply the prestressing wires to the outside surfaces as well as to the inside for added stability against bucking due to external loads.

The wires mainly followed the ruling lines of the hyperbolic paraboloidal surfaces. In the case of some shells whose rulings intersected at sharp angles it was necessary to also apply wires that followed the bisectors of the obtuse angle between the ruling lines. These wires were an exception in that they followed a curved path.

For the prestressing to be completely effective, it was necessary that the bottom edge of a shell be able to move

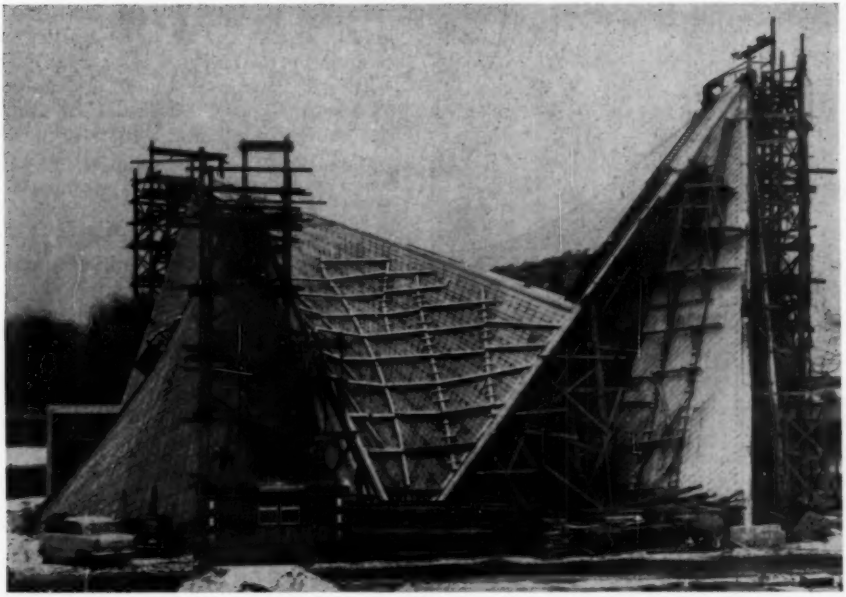


Fig. 4—The pavilion shortly before completion showing the numerous prestressing wires

a few centimeters (without changing its inclination) over the foundation during the prestressing process. This condition called for special measures at various points, particularly for those shells resting on the foundation. The substructure of these shells was initially separated from the foundation by two layers of bituminous felt (which served later for waterproofing); this permitted relative movement between substructure and foundation until after prestressing was completed, when the whole structure was joined

Fig. 5—Prestressing wires for a rib before being enclosed with the formwork. This particular rib was fitted only with wires for producing torsional prestressing, (eight wires running on the circumference of a cylinder 35 cm in diameter formed by a series of iron hoops). The 9-mm wires look thicker here since they have been coated with a preparation to prevent bonding with the concrete

Fig. 6—Tensioning a presiressing wire with the "Strabed" jack. Note the special anchorage devices extending from the rib

to the foundation by the addition of concrete. Without this measure some of the prestressing forces would have been taken up by the foundations and the stresses introduced in the shell would not have had the right direction.

The ribs were also prestressed in three distinct ways. Compressive prestress had to be produced by straight wires so that the ribs could take up the tensile stresses as measured in the model tests. Second, some ribs had to be given a bending prestress by means of wires following a parabolic path in a plane through the axis. This was necessary in particular in the case of two ribs at the entrance and exit to which only one almost flat shell abutted. Third, torsional prestresses were introduced in a number of ribs (Fig. 5) to counteract the torsional moments imparted to them by the prestressed abutting shells. Although the concrete of the ribs was capable of taking up these torsional moments, it was desirable to eliminate them to insure centralized prestress in the shells. (It may be that this is the first application of torsional prestressing in concrete.)

All wires were tensioned by a method devised by the contractor Strabed. The main feature of this method is the possibility of tensioning wires between anchorages fitted to the structure in advance (Fig. 6).

The exterior prestressing wires were left exposed after being given a coating of special waterproof paint and a final coating of aluminum paint. The prestressing wires on the inside shell surfaces were concealed by a sound absorbent layer.

Acknowledgments

The erection of this unique structure—being an exhibition building that could be considered a "temporary" structure—has offered the industry an ideal opportunity to gather information and experience on methods of construction which certainly point the way to interesting and wide possibilities.

The structure was designed by Le Corbusier and Y. Xenakis. The method of construction was proposed and translated into reality by N. V. "Strabed": Société de Travaux en Béton et Dragages, H. C. Duyster, director. Professor C. G. J. Vreedenburgh, Delft Technische Hogeschool, served in a consulting capacity on the shell design. Tests on scale models were made by A. L. Bouma and F. K. Ligtenberg in the T.N.O. Institute at Rijswijk (Netherlands) and the Stevin Laboratory at Delft.

McDaniel joins Denie's staff

John E. McDaniel, Jr., has joined the firm of John A. Denie's Sons Co., Memphis, as a sales engineer. Mr. McDaniel has been a concrete engineer with Marquette Cement Manufacturing Co., Memphis, for the past 11 years.

LOOKING AHEAD

June 16-22, 1960—Third International Congress of Precast Concrete Industry, Sankt Erikshallen, Stockholm, Sweden

June 20-22, 1960—FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

June 20-24, 1960 — Annual Meeting, American Society of Engineering Education, Purdue University, Lafayette, Ind.

June 26-July 1, 1960 — Annual Meeting and Apparatus Exhibit, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

June 27-July 1, 1960—Sixth Congress, International Association for Bridge and Structural Engineering, Stockholm, Sweden

July 6-8, 1960—Cooper Union-ASCE-ASEE Conference on Civil Engineering Education, University of Michigan, Ann Arbor

July 18-22, 1960—Thin Shell Concrete Structures Study Course, Case Institute of Technology, Cleveland, Ohio

July 10-22 and July 24-August 5, 1960—Engineering Seminars on Atomic Shelters, Pennsylvania State University, University Park, Pa.

July 11-18, 1960 — Second World Conference on Earthquake Engineering, Science Council of Japan, Tokyo and Kyoto, Japan.

July 17-21, 1960—RILEM International Symposium on Concrete and Reinforced Concrete in Hot Countries, Israel Institute of Technology, Haifa, Israel

Aug. 15-26, 1960 — Summer Institute on Nondestructive Testing, Sacramento State College, Sacramento, Calif.

Sept. 27-30, 1960 — 6th Annual Convention, Prestressed Concrete Institute, Statler-Hilton Hotel, New York, N.Y.

Oct. 10-13, 1960 — American Mining Congress, Metal Mining and Industrial Minerals Convention and Exposition, Las Vegas, Nev.

Oct. 31-Nov. 1-2, 1960—13th Regional Meeting, American Concrete Institute, Pioneer Hotel, Tucson, Ariz.

Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

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Architects and Engineers: Connell, Pierce, Garland & Friedman, Miami, Florida
General Contractor: Graves Construction Company, Miami, Florida

Concrete Reinforcing
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Along our seaboards where salt air often corrodes construction materials, reinforced concrete is used extensively for buildings of all types and sizes. At the Allapattah Baptist Church Educational Building in Miami, Florida, reinforced concrete was specified for its outstanding corrosion resistance, as well as its *lower construction costs, lower maintenance costs, and greater fireproofing qualities.*

Investigate the superior design and cost-saving qualities of this extremely flexible construction medium.

Positions and Projects

President Kelly speaks at CSI annual meeting

ACI President Joe W. Kelly, professor of civil engineering, University of California, Berkeley, was guest speaker at the fourth annual meeting of the Construction Specifications Institute in Palo Alto, Calif.

McCoy and Pregnoff honored for TAC work

The ACI Board of Direction and Technical Activities Committee expressed their thanks at the March annual convention to Walter J. McCoy, director, Research Laboratory, Lehigh Portland Cement Co., Coplay, Pa., for his outstanding leadership as chairman of TAC for the last 2 years and to M. V. Pregnoff, Pregnoff and Matheu, San Francisco, for his numerous valuable technical contributions to the work of the Technical Activities Committee.

Clyde E. Kesler, professor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, is chairman of the committee for 1960. Institute President Joe W. Kelly, professor of civil engineering, University of California, Berkeley, is ex officio member of the committee with William A. Maples, ACI secretary-treasurer, acting as secretary of the committee.

G. E. Burnett, assistant chief engineer, Engineering Laboratories, U. S. Bureau of Reclamation, Denver, and E. A. Finney, director, Highway Research Laboratory, Michigan State Highway Department, East Lansing, were reappointed members for 2-year terms. Harry Ellsberg, consultant, Giffels and Rossetti, Detroit, was appointed for a 2-year term. Mr. McCoy is serving a 1-year term to fill the unexpired term of Clyde E. Kesler, newly appointed committee chairman.

Bryant Mather, chief, Special Investigation Branch, Concrete Division, Waterways Experiment Station, Jackson, Miss., and J. J. Shideler, manager, Products and Applications Section, Portland Cement Association Research and Development Laboratories, Skokie, Ill., are also serving on the TAC.

Technical committee chairman appointments

The ACI Board of Direction has reappointed Hubert Woods, director of research, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., for a 1-year term as chairman of Committee 201, Durability of Concrete in Service. William R. Waugh, chief of the concrete branch, Engineering Division, Civil Works, Office of the Chief of Engineers, Department of the Army, Washington, D.C., was reappointed for a 1-year term as chairman of Committee 621, Aggregates.

E. P. Rippstein, chief engineer, Construction Products, Laclede Steel Co., St. Louis, was appointed chairman of Committee 315, Detailing Reinforced Concrete Structures, for a 3-year term.

Board of Direction discharges two ACI technical committees

Two ACI technical committees, having accomplished their respective missions, were discharged by the Institute Board of Direction at its March meeting in New York: Committee 605, Hot Weather Concreting; and Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete.

Committee 605, Hot Weather Concreting

Organized in 1955, this committee was given the assignment to review information on the effect of hot weather on the characteristics of fresh and hardened concrete, and to develop a recommended practice for controlling ingredients, batching, mixing, transporting, placing, protecting, and curing concrete during hot weather.

A progress report, "Hot Weather Concreting Problems," was presented at the 1957 annual convention and published in the May, 1957, JOURNAL (*Proceedings* V. 53).

The ACI Standard "Recommended Practice for Hot Weather Concreting (ACI 605-59)" was adopted at the 55th annual convention and ratified by letter ballot in June, 1959. The proposed standard appeared in the November, 1958, JOURNAL (*Proceedings* V. 53). The Standard (ACI 605-59), adopted without change, has been printed separately by the Institute.

Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D.C., was chairman of the committee.

D. L. Bloem, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D.C., was secretary of the committee. Members included: Clayton L. Davis, Universal Atlas Cement Division, United States Steel Corp., New York; John L. Goetz, Southwestern Portland Cement Co., Los Angeles; Nomer Gray, Ammann and Whitney, New York; Ernst Gruenwald, Lone Star Cement Corp., New York; H. F. Hedderich, Pittsburgh Testing Laboratory, Pittsburgh; E. L. Howard, Pacific Cement and Aggregates, Inc., San Francisco; Harry F. Irwin, Warner Co., Philadelphia; Harry H. Mitchell, Spartanburg Concrete Co., Spartanburg, S.C.; George H. Nelson, Law Engineering Testing Co., Atlanta; George L. Otterson, Omaha; Perry H. Petersen, The Master Builders Co., Cleveland; Emil Schmid, Sika Chemical Corp., Passaic, N.J.; John H. Swanberg, Minnesota Department of Highways, St. Paul; and Lewis H. Tuthill, California State Department of Water Resources, Sacramento.

Rounding out the committee were: I. L. Tyler, Portland Cement Association, Skokie, Ill.; Byron P. Weintz, Consolidated Rock Products Co., Los Angeles; John J. White, Parsons, Brinckerhoff, Quade and Douglas, Inc., New York; and C. E. Wuerpel, Marquette Cement Manufacturing Co., Chicago.

Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete

This committee was given the assignment to review and bring up to date the standard "Recommended Practice

Looking Ahead to

1961 in St. Louis

ACI's Technical Activities Committee is already shaping up program plans for the 1961 convention to be held in St. Louis, February 20-23.

Those who intend to offer papers for presentation at the 57th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1960, and final manuscripts of accepted papers will be due Jan. 1, 1961.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters now about the papers you want to see on the 1961 program

for Measuring, Mixing, and Placing Concrete (ACI 614-42)" published in the June, 1945, JOURNAL.

Originally organized in 1936, the committee was discharged after completion of the 1942 standard and was reorganized in 1956.

A revised standard "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)" was adopted at the 55th annual convention and ratified by letter ballot in June, 1959, to supersede ACI 614-42. The proposed standard was published in the November, 1958, JOURNAL (*Proceedings* V. 55) with errata published in the July, 1959, JOURNAL (*Proceedings* V. 56). The standard has been printed separately by the Institute.

Lewis H. Tuthill, California State Department of Water Resources, Sacramento, was chairman of the committee. Working on the committee were: Alvin W. Brust, Washington University, St. Louis; H. F. Hedderich, Pittsburgh; E. L. Howard, Pacific Cement and Aggregates, Inc., San Francisco; John J. Manning, The Concrete Industry Board, New York; Harry H. McLean, New York Department of Public Works, Albany; and William T. Neelands, Sverdrup and Parcel Engineering Co., Bangkok, Thailand.

Others working on the committee were: Bert Noble, mechanical engineer, Paso Robles, Calif.; T. J. Reading, Corps of Engineers, Omaha; M. R. Smith, Office Chief of Engineers, Department of the Army, Washington, D. C.; J. W. Winkworth, J-W Materials, Inc., Napoleon, Ohio; L. P. Witte, U. S. Bureau of Reclamation, Denver; and D. K. Woodin, East Bay Municipal Utility District, Oakland, Calif.

ACI publishes bibliography on evaluation of strength tests

ACI has published its Bibliography No. 2, "Evaluation of Strength Tests of Concrete." This new bibliography lists and annotates selected articles appearing in available technical publications issued from 1924 to 1958, dealing specifically with compression tests of concrete, variations in test results, and evaluation of tests. The bibliography was compiled as part of the work of ACI Committee 214, Evaluation of Results of Strength Tests of Concrete.

The new ACI bibliography is in 8½ x 11-in. format, saddle-stitched, and punched for easy insertion in a three-ring binder. Price per copy is $2.00.

Southern California Chapter elects Butler '60 president

About 100 members and guests attended the second annual meeting of the ACI Southern California Chapter on March 30 at the Rodger Young Auditorium in Los Angeles.

Dewain R. Butler

Results of balloting for officers named Dewain R. Butler, president of Integrated, Inc., Los Angeles, as president of the chapter for the ensuing year with Byron P. Weintz, chief engineer, Consolidated Rock Products Co., Los Angeles, vice-president. Elected directors for 3-year terms were: Henry W. Smith, president, Bell Construction Co., Los Angeles, and C. Taylor Test of Riverside Cement Co., Los Angeles.

Elected to the committee on nominations were: chairman, Harald Omsted, chief structural engineer, Board of Education, Los Angeles; W. C. Bryant, field engineer supervisor, Portland Cement Association, Los Angeles; Edward W. Twining, Twining Laboratories, Long Beach, Calif.; and William T. Wheeler, Wheeler and Gray, Los Angeles.

J. L. Peterson, retiring chapter president, presided and reported on the highlights of the 56th annual convention of the Institute, held in New York in mid-March.

Long-time ACI convention attendees!

ACI annual meetings date back to 1905 when the first convention was held in Indianapolis with 761 in attendance.

Past President Herbert J. Gilkey, Iowa State University, Ames, reports that up to the 56th annual convention in New York last March, he had an unbroken attendance record at ACI annual meetings for the past 26 years.

Harry C. Delzell, Concrete Reinforcing Steel Institute, Chicago, has established a commendable record in that he has been in attendance at every ACI annual convention that has been held for the past 22 years and also has attended every regional meeting that has been held during the same period.

Can you top these records? Let us hear from long-time ACI convention attendees!

The meeting honored as guest and speaker, the 1960 national president of the Institute, Joe W. Kelly, professor of civil engineering, University of California, Berkeley. Speaking on the theme "What is ACI?" President Kelly presented an enlightening word picture of the make-up, work program, and various activities of the Institute, now numbering a membership of over 10,200, 18 percent of which represents countries outside the United States and Canada. He spoke of the JOURNAL as the mainspring of ACI's program for service to its members and to the public in providing up-to-date technical information and making it available for use. He cited the 52 active technical committees working on various subjects and problems through the efforts of about 900 committee members and called attention to the wide use and acceptance of ACI standards and recommended practices. He spoke of the potential importance of ACI chapters, of which the Southern California Chapter was the first organized, and announced the approval of the fourth chapter in Michigan.

Special guests present to honor President Kelly included: Edward T. Telford, assistant state highway engineer; Newton H. Temblin, road commissioner, Los Angeles County; John C. Monning, assistant superintendent of building, City of Los Angeles; Jack N. Sparling, president, Structural Engineers Association of Southern California; Bernard S. Harder, chairman, Structural Group, Los Angeles Section, ASCE; John B. Howe, vice-chairman, Southern California District Council, ASTM.

W. F. Norton, president, Western Reinforcing Steel Fabricators Association, assisted by F. S. Clough, WRSFA manager, made a solid contribution to the occasion by presenting Mr. Peterson a weighty gavel, beautifully fabricated of bars of reinforcing steel. This appropriate emblem of office was turned over to the new vice-president, Byron Weintz, acting for President Butler.

Sam Hobbs, secretary-treasurer of the chapter, reported briefly on the status of the chapter, touching on finances, membership, and activities.

Michigan Chapter holds first official meeting

The first official meeting of the ACI Michigan Chapter was held in Detroit on April 13. William Maples, secretary-treasurer of the Institute, announced that the petition for the chapter had been accepted by the Board of Direction at the time of the 1960 annual convention in New York City. The Michigan Chapter will embrace all of the state of Michigan and Lucas County of Ohio, where the principal city is Toledo.

James E. Hampton, technical director, American Aggregates Corp., Detroit, was principal speaker for the luncheon meeting. Mr. Hampton, who was formerly in charge of the soil and concrete testing sections for the Detroit district

of the Corps of Engineers, discussed and showed a number of slides on the subject of airfield pavements. While he was with the Corps, the expansion of four airfields to receive SAC bombers was undertaken. The pictures showed construction operations on some of these fields, demonstrating the preparation of the subbase, batching plant setup, paving operations, and construction of overlay pavements.

McHenry reappointed chairman of ACI Standards Committee

The Board of Direction has approved the reappointment of Douglas McHenry, director of development, Portland Cement Association Research and Development Laboratories, Skokie, Ill., as chairman of the Standards Committee for a 1-year term. Walter J. McCoy, director, Research Laboratory, Lehigh Portland Cement Co., Coplay, Pa., and Thor Germundsson, staff adviser, Portland Cement Association, Chicago, have been appointed as members of the committee for 3-year terms.

Continuing as members of the Standards Committee are: Bruce E. Foster, materials engineer, National Bureau of Standards, Washington, D.C.; Joe W. Kelly, professor of civil engineering, University of California, Berkeley; Thomas B. Kennedy, chief, Concrete Division, Waterways Experiment Station, Jackson, Miss.; and James A. McCarthy, professor of civil engineering, University of Notre Dame, Notre Dame, Ind. William A. Maples, Institute secretary-treasurer, acts as secretary of the committee.

Seventh annual concrete conference at North Dakota

The seventh annual concrete conference conducted by the University of North Dakota, Grand Forks, was held on March 28-29 with the cooperation of ACI, the Portland Cement Association, and local chapters of various organizations. About 170 architects, engineers, contractors, producers, and students attended the conference.

The first afternoon session dealt primarily with recent developments and applications of new thin-shell roof shapes. The PCA film "New Shapes in Concrete" opened the program. This was followed by discussions of construction methods from the structural viewpoint presented by Ralph Spears, PCA regional structural engineer, Salt Lake City, and the contractor's viewpoint presented by Gordon Madsen of George Madsen Construction Co., Minneapolis. Mr. Madsen discussed the economies of hyperbolic paraboloidal shell roofs. He concluded by presenting a film showing the construction of hyperbolic paraboloids at a shopping center in Bloomington, Minn., constructed by his company.

Kenneth D. Cummins, ACI technical director, opened the morning session on March 29, giving a talk on concrete mixes for the job. J. J. Shideler, development engineer, PCA Research and Development Laboratories, Skokie, Ill., discussed mixing, placing, and properties of lightweight structural concrete. Norman Henning, vice-president, Twin City Testing and Engineering Laboratory, Inc., St. Paul, spoke on bonding agents for concrete, stressing primarily the use of the new epoxy resins. His talk was followed by a question and answer and discussion period led by the three speakers.

In the final afternoon session Mr. Spears discussed precast wall panels. The PCA film "Concrete '59" was presented and followed by a talk on the design and construction procedures of floors by Lyman S. Bray, chief of the material and airfield pavement design section, U. S. Corps of Engineers, Omaha. A discussion period followed and rounded out the program.

Dean E. L. Lium of the College of Engineering, University of North Dakota, was general chairman for the conference. He was assisted in the preparation of the program by other faculty members and Robert Randall

of the Minneapolis office of the Portland Cement Association and other PCA engineers.

Amirikian to represent ACI at Stockholm precast congress

A. Amirikian, special structures consultant, Bureau of Yards and Docks, Department of the Navy, Washington, D.C., will act as official ACI delegate at the Third International Congress of the Precast Concrete Industry in Stockholm, June 16-22, 1960.

The congress has been organized by the Bureau International du Béton Manufacturé (BIBM) in Brussels in cooperation with the Swedish precast concrete industry's branch association, Cementvarufabrikernas Riksförbund (CR) in Stockholm.

RILEM symposium to be held in Haifa in July

Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions (RILEM) will conduct a symposium on "Concrete and Reinforced Concrete in Hot Countries" in Haifa, Israel, July 17-19 at the Israel Institute of Technology.

Rahel Shalon, chief of the Building Research Station at the Technion–Israel Institute of Technology, Haifa, is president of RILEM, and is serving as chairman of the organizing committee. Professor Shalon has been a member of ACI since 1947 and has twice contributed technical papers for publication in the JOURNAL.

Four ACI members are among several prominent European concrete authorities participating in the program. F. M. Lea, director of building research station, Department of Scientific and Industrial Research, Herts, England, will chairman the first materials session. Dr. Lea is a member of ACI Committee 115, Research.

R. L'Hermite, director, Laboratories du Batiment et des Travaux Publics, Paris, will act as chairman of the materials session on July 18. Mr. L'Hermite is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension. Inge Lyse, professor of reinforced concrete and director of the concrete laboratory, Norway's Institute of Technology, Trondheim, will chairman the session on durability. Long active in ACI, Professor Lyse has contributed numerous papers for JOURNAL publication. He is currently a member of Committee 115, Research, and Committee 201, Durability of Concrete in Service.

E. Torroja, director of Instituto Technico de la Construcction y del Cemento, Madrid, will act as chairman of the structures session. Professor Torroja was elected an honorary member of ACI in 1959 for his outstanding contributions to the concrete field. He is currently a member of Committee 115, Research. Other technical sessions will cover pavements and foundations, and recommendations for further research.

The official languages of the symposium are English and French. Summary translations of proceedings from one language into the other will be provided.

Case schedules short course on thin shell structures

Case Institute of Technology, Cleveland, has scheduled a one-week summer study course, July 18-22, on thin shell concrete structures. The course is designed for engineers and architects who would like to know more about the theory and design of the new shapes currently being built throughout the country and abroad. The course includes the analysis and design of domes, hyperbolic paraboloids, cylindrical shells, and folded plates. The presentation will develop the necessary theory and design principles for typical structures.

The primary objective of the course is to develop an understanding and appreciation of the theory and design applications of thin shell structures.

The course will include lectures devoted to the development of the theory for the various thin shell structures and periods for the solution of typical problems and discussion of questions stimulated by the lectures.

Details are available from John B. Scalzi, Department of Civil Engineering, Case Institute of Technology, Cleveland.

Davis named to key post at Universal Atlas Cement

Clayton L. Davis has been named director of Quality and Technical Services Division of Universal Atlas Cement Division of U. S. Steel, New York.

Following several years of experience in the physical testing of cement for Huron Portland Cement Co. and Michigan State Highway Department, he joined Universal Atlas Cement Co. at Chicago in 1930 as engineer in its inspection bureau. Advancing to a number of responsible posts within the organization, Mr. Davis was appointed technical service director in 1955.

Long active in a number of technical societies, he is a former director of ACI and is currently chairman of Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Griffith joins Anderson firm

K. J. Griffith, formerly associated with ARC Engineering in Homebush, N.S.W., Australia, is now chief engineer of A. J. Anderson Pty., Ltd., North Sydney, N.S.W., Australia.

"Open house" builds acceptance for concrete masonry home construction

To increase acceptance of concrete masonry in new home construction in the Lansing, Mich., area, Standard Block & Supply Co. recently staged an "open house" to demonstrate advantages of the material to mortgage lenders, architects, builders, and potential home buyers.

Scene of the demonstration was an attractively designed house featuring the "all-in-one" wall concept which permits use of the concrete block wall itself for both exterior and interior finish, eliminating the need and expense for additional finish materials. Insulation for the walls was provided by a water-repellent vermiculite masonry fill, developed by Zonolite Co., which was poured directly into the cores of the block.

The home was designed by Gordon Fineout, Lansing designer, for Standard's plant superintendent, Maynard Simmons. Al and Stan Powell of Lansing were the contractors. Located in the Tacoma Hills section of Okemos, a Lansing suburb, the 2000-sq ft house was built at a cost of approximately $14 per sq ft, compared to an average of $18 to $20 per sq ft for conventional construction in the area.

Built on slab, the one-level house incorporated the economy of post and beam construction with comparatively inexpensive modern concrete block walls. On the interior of the house, the posts and beams served as trim, and the exposed block provided the wall decor.

Insulated concrete masonry wall allows the use of exposed masonry finish for both exterior and interior surfaces of home

ASTM symposium to probe basic materials sciences

Analyses of the present state of knowledge of several facets of materials science will highlight the American Society for Testing Materials annual meeting at Chalfonte-Haddon Hall, Atlantic City, June 26-July 1, 1960.

The society's new Division of Materials Sciences has arranged a full-day program for Monday, June 27. A "Symposium on Recent Progress in Materials Sciences" will convene in the morning. The afternoon session will focus on a single property—strength.

The ASTM annual meeting at which an attendance of more than 3000 is expected, will include 35 to 40 technical report sessions spread over the 5-day period. Eight to 900 technical committees and their subcommittees will hold meetings during the week and there will be an exhibit of testing and scientific apparatus and laboratory supplies, and a photographic exhibit in conjunction with the meeting.

1960 officers elected by Vermiculite Institute

L. G. McDiarmaid of Vancouver, B.C., Canada, was elected to the presidency of the Vermiculite Institute, Chicago, at the 19th annual meeting of the organization, March 19-24, at Chandler, Arizona.

C. H. Wendel of Los Angeles and L. K. Irvine of Salt Lake City, were named to the board of directors. Edward R. Murphy was returned as managing director and Walter J. Bein of Chicago was reelected treasurer.

New developments in vermiculite and allied products of interest to the construction industry were the main topics of discussion at the meeting. C. R. Ball, chairman of the concrete committee, presented a new data sheet and short form specification for a vermiculite roof system, recently fire-rated at 2 hr, that combines insulation, sound control, and fire protection in one package. He also announced that the Army Corps of Engineers, Washington, D.C., has published a comprehensive new specification for vermiculite concrete roof decks over steel and various formboards. L. J. Lambert, chairman of the insulation committee, reviewed recent tests at Pennsylvania State University of concrete block walls with the cores filled with water-repellent vermiculite masonry fill.

TEC merges with Wiss and Associates

The officers of The Engineers Collaborative, consulting structural engineering firm of Chicago, have announced the merger of their organization with that of Wiss and Associates. The merger of the two firms will broaden and compliment the services already offered by TEC to architects, engineers, and others in the construction industry. Mr. Wiss becomes an executive member of TEC.

Floyd Concrete opens Brunswick plant

Floyd Concrete Co., Savannah, Ga., has opened a new $250,000 ready-mixed concrete plant in Brunswick, Ga. The new plant, with 12 employees, will have five mixer trucks and a 100 cu yd per hr capacity. Ralph Flanders, formerly vice-president-general manager, has been named president of the company.

Leap Associates to open plant in Mexico City

Harry Edwards, president of Leap Associates, Lakeland, Fla., has announced the completion of negotiations for a new prestressing plant to be located in Mexico City.

William D. Givens, research and project engineer at Leap, will spend several months in Mexico to set up the new plant and train personnel. The plant will supply the Mexico City area

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

Individual Members (U.S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$20.00**
Individual Members (All other foreign countries)	**16.00**
Corporation Members	**65.00**
Junior Members—nonvoting (under 28)	**10.00**
Contributing Members	**135.00**
Student Members—nonvoting (under 28)	**5.00**

Please enclose remittance with application (cut here)

Board of Direction, American Concrete Institute **Date**_____
P. O. Box 4754, Redford Station
Detroit 19, Michigan

The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter) _____

_____

For Corporation Membership, ACI representative will be _____

Date of Birth (Juniors and Students only) _____
Year Month Day

College or University attending (Students only) _____

_____ _____
(Date of graduation) (Proposed by)

Signature_____

For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

MEMBER CLASSIFICATION

JOB TITLE _____

OCCUPATION (Check the one most applicable)

☐ Arch ☐ Engr ☐ Construction Supervision ☐ Plant Management or Supervision ☐ Teaching ☐ Student ☐ Other (please state) .....

EMPLOYER

☐ Architect ☐ Contractor ☐ Consulting Engr ☐ Engr Firm ☐ Manufacturer or Producer (specify product) .....

Government ☐ Fed ☐ State ☐ County ☐ City ☐ Educational Institution ☐ Commercial Testing Laboratory ☐ Public Utility ☐ Trade Assn ☐ Library ☐ Other (please state) .....

PRINCIPAL RESPONSIBILITY (Check the one most applicable)

☐ Design ☐ Construction ☐ Consulting ☐ Purchasing ☐ Sales ☐ Advertising ☐ Research ☐ Administrative (state position) .....

☐ Other (please state) .....

Do you ☐ Specify ☐ Authorize ☐ Recommend, purchase of materials or equipment?

with cored slabs, joists, beams, columns, and other elements.

Other changes in the organization include the appointment of J. Robert Lundy to the position of sales manager and J. Harrison Smith as public relations and coordination director.

Dobbins awarded USCC citation

Cris Dobbins, president of Ideal Cement Co., Denver, was among 62 men from throughout the country cited for "outstanding contributions to the construction industry and to the advancement of the free enterprise system" at the National Construction Industry Conference in Washington, D.C., March 10-11. The conference was sponsored by the United States Chamber of Commerce.

BRI establishes building science education fund

The Building Research Institute, National Academy of Sciences–National Research Council, Washington, D.C., recently announced the establishment of a new Building Science Education Fund, to be used for the stimulation of a program for encouraging highly creative people to pursue building research activities in colleges and universities.

Milton C. Coon, Jr., BRI executive director, said, "The BRI board of governors has appropriated $2000 from Institute funds to launch this enterprise, one which we feel will have far-reaching effects in encouraging college and university research workers to focus their attention on some of the major problems that need solving in our advance toward better buildings."

It is expected that the fund will grow rapidly through contributions from all of the various disciplines which go to make up the broad complex of building design, construction, operation, and maintenance. Inquiries regarding the fund may be directed to Milton C. Coon, Jr., executive director, BRI, National Academy of Sciences–National Research Council, 2101 Constitution Ave., Washington 25, D.C.

Wilbraham joins Rackle sales staff

James O. Wilbraham has been appointed district sales manager of George Rackle and Sons Co., Cleveland. He will act in a sales and liaison capacity with northeastern Ohio architects, engineers, and contractors.

PCA opens new regional and district offices

The Portland Cement Association has announced major changes in its field promotion organization including establishment of a new regional and new district office. The states of Oklahoma, Arkansas, Louisiana, and Texas are being joined together in a new south central region with headquarters at Austin, Tex. Thomas D. Shiels has been named south central regional manager. Ted L. Edwards succeeds Mr. Shiels as district engineer in Texas.

Edward D. Thorson has been appointed full-time regional manager for the Rocky Mountain region with headquarters in Denver.

Bernard C. Smith has been named New Mexico district engineer with offices at Albuquerque. Jack Y. Barnes has been named district engineer for Colorado and Wyoming, with offices in Denver. Norman D. Baumgart has been named district engineer for Montana with offices in Helena.

Errata

The following corrections should be made in "Design of L-Shaped Columns with Small Eccentricities," by L. S. Muller, which appeared in the December, 1959, ACI JOURNAL.

p. 494—in the last line the expression "$Q = 0.08$" should read "$Q = 0.8$".

F. Eugene Seaman

F. Eugene Seaman, chief research engineer in charge of the laboratory of Lock Joint Pipe Co., Wharton, N.J., recently died at the age of 43.

Mr. Seaman received his BS in civil engineering from Ohio State University in 1939, and was employed by the testing and research laboratory of the Ohio State Highway Department until 1941.

After 5 years service with the U. S. Army Corps of Engineers, where he was engaged in airfield construction, he returned to the Ohio State Highway Department laboratory in 1946 and studied the behavior of confined concrete under stresses approaching the compression limits of structural steel.

He received a second engineering degree from Ohio State University in 1947.

Since 1948, Mr. Seaman had been chief research engineer in charge of the laboratory of Lock Joint Pipe Co. He has conducted numerous studies in cement, aggregates, and high tensile strength steel that entered into the making of prestressed concrete products both with respect to lineal and circular prestressed items with major emphasis on the latter.

Epstein Memorial Award winners announced

Three members of the University of Illinois Department of Civil Engineering staff have been selected as the 1960 winners of the A. Epstein Memorial Award.

The recipients are James E. Stallmeyer, associate professor; J. L. Merritt, assistant professor, and Steven J. Fenves, instructor. All of the awardees are members of ACI.

Funds for the award were contributed by Raymond and Sidney Epstein, Chicago, University of Illinois civil engineering graduates in 1938 and 1943, in memory of their father who died in 1958. A. Epstein, a CE graduate in 1911, had established the original fund in 1955. The sons are continuing the operation of the consulting engineering firm A. Epstein and Sons, Inc., which was founded in 1947.

The awards are made to outstanding young staff members having the rank of instructor, research associate, assistant professor, and associate professor, based on the personal achievements of the candidates, scholarship, professional standing, service to the department, and the general over-all contribution of the men to the prestige of the civil engineering department and that of the university.

AGC presents safety awards

Three ACI member firms were among contractors awarded safety citations by the Associated General Contractors of America during the 41st annual convention of the association in San Francisco.

Holmes and Narver, Inc., Los Angeles, took second place in the Group A classification (over 500,000 hr exposure) in both the "building division" and the "heavy division."

Pulte-Strang, Inc., Ferndale, Mich., took first place in the Group C classification (over 100,000 hr exposure) in the building division.

Second place for the "Best Ten-Year Record" in the heavy division was awarded Maxon Construction Co., Inc., Dayton, Ohio.

Dvorak promoted

J. J. Dvorak has been named assistant division head of the Technical Division at Humble Oil and Refining Co.'s Baytown, Tex., refinery.

Forbrich named vice-president of Green Bag Cement

Louis R. Forbrich has been named vice-president of Green Bag Cement Co., a wholly-owned subsidiary of Pittsburgh Coke and Chemical Co., Pittsburgh. He will continue as general manager of the company.

Honor Roll

Jan. 1 — Apr. 30, 1960

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

In the past 4 months nearly 200 members have joined the Honor Roll by proposing new members for the Institute. Add your name to the Honor Roll next month. Let's keep the membership trend spiraling!

Alfonso Marin E. 33
Samuel Hobbs 29
Harry Ellsberg 15
Walter H. Price 12
Robert P. Witt 11
Joaquin Spinel L. 10
Faraj Tajirian 10
Gene M. Nordby 9
J. Raymond Watson 9
James Chinn 7
Napoleon Ferrer G. 7
James A. McCarthy 7

Roger H. Corbetta 6
Alfonso Golderos 6
Arturo Guevara 6
T. C. Kavanagh 6
John C. McCoe 6
Jose Luis Montemayor 6
George B. Southworth 6
R. H. Wildt 6
Milton H. Zara 6
Pedro M. Bassim 5
W. S. Cottingham 5
John E. Heer, Jr. 5
Antonio A. Henson B. 5
J. Karni 5
Adrian Pauw 5
H. C. Pfannkuche 5
C. P. Siess 5
J. F. Toppler 5

Felix Colinas Villoslada.. 4
John G. Dempsey 4
Ward W. Engle 4
Phil M. Ferguson 4
William W. Karl 4
Joe W. Kelly 4
Narbey Khachaturian ... 4
Henry A. Lepper, Jr. 4
Jack Longworth 4
Willard A. Oberdick 4
Gerald F. Paulson 4
Pastor B. Tenchavez 4
John Adjeleian 3
James E. Amrhein 3
S. A. Asplund 3
Amos Atlas 3
Salvatore Azzaro 3
J. E. Backstrom 3
Ira M. Beattie 3
George B. Begg, Jr. 3
Luther E. Bell 3
M. R. Berretti 3
Dan E. Branson 3
Martin R. Brown, Jr. 3
Allen H. Brownfield 3
Vincent R. Cartelli 3
Alan Carter 3
Fong C. Chen 3
G. L. Cubbison 3
Edward J. Curtin 3
Ramzi A. Dabbagh 3
Evan Savours David 3
Fernando de Angulo 3
Roger Diaz deCossio 3
H. C. Delzell 3
James N. De Serio 3
H. J. Dickinson 3
Peter E. Ellen 3
Marco Estrada 3
Arthur Feldman 3

Benjamin P. Felix 3
R. J. Fisk 3
Russell S. Fling 3
Joseph J. Fox 3
Frank D. Gaus 3
Hans Gesund 3
H. J. Gilkey 3
Alfred G. Graves 3
Fabian Guerra 3
James E. Halpin 3
Kenneth Hansen 3
Norman E. Henning 3
Roy Holte 3
Robert H. Hopwood 3
Kenneth M. Huber 3
G. M. Idorn 3
E. H. Johnson 3
H. Alan Johnson 3
Robert J. Kadala 3
Karl Kaspin 3
Clyde E. Kesler 3
Milo S. Ketchum 3
F. R. Killinger 3
John C. King 3
Arthur A. Klein 3
E. V. Konkel 3
George Kostro 3
William J. Krefeld 3
Simon Lamar 3
Cecil M. Langford 3
L. R. Lauer 3
Howard Losey, Jr. 3
John T. McCall 3
W. J. McDonald 3
John M. McNerney 3
M. F. Macnaughton 3
Luis F. Magrina 3
Roman Malinowski 3
E. M. Markell 3
Ignacio Martin 3

N. D. Morgan 3
W. E. Moulton 3
Poul Nerenst 3
Frank J. Oleri 3
Gregorio Ortega 3
Miguel Angel Ortiz Agudin 3
William E. Parker 3
John D. Paterson 3
T. Paulay 3
E. A. Peterson 3
Russell Porter 3
Z. Przygoda 3
Frank A. Randall, Jr. ... 3
Abdur-Rahman S. Rasul. 3
Ilmar Reinart 3
Salvador Rodriguez 3
Howard J. Rosenberg ... 3
H. Rusch 3
R. S. Sandhu 3
E. A. Sanford 3
Herbert A. Sawyer, Jr... 3
John B. Scalzi 3
C. H. Scholer 3
Morris Schupack 3
H. M. Schwartz 3
Harold J. Sexton 3
M. F. A. Siddiqui 3
A. L. Small 3
John D. Smith 3
S. Szalwinski 3
F. K. Taskin 3
W. R. Thessman 3
J. Antonio Thomen 3
Alex Tobias 3
Rogelio Bonilla Torres .. 3
D. A. Van Horn 3
Ellis S. Vieser 3
Sam Walden 3

Carl Weber 3
Cedric Willson 3
J. A. Wineland 3
T. W. Wood 3
Mark R. Woodward 3
Jayantilal S. Alagia 2
Arthur H. Andersen 2
Carlos Asturias P. 2
Charles O. Baird, Jr. 2
John E. Bower 2
D. Campbell-Allen 2
Francisco Castano Hernandez 2
Guillermo Castellanos G. 2
M. H. Chapman, Jr. 2
William G. Corley 2
Clayton M. Crosier 2
Graham Earle 2
G. Grenier 2
Martin J. Gutzwiller ... 2
Robert B. Harris 2
Gregorio Hernandez 2
William W. Hotaling, Jr. 2
Raja A. Iliya 2
Fred W. Jacobs 2
Carl B. Johnson 2
Ralph F. Jones 2
Oscar Latorre M. 2
Howard R. May 2
Abel Moreno Przespolewski 2
R. R. Neal 2
Carlos Luis Nebreda 2
Carlos Isunza Ortiz 2
Raoul E. Pallais 2
Roberto E. Prata Lou ... 2
Luis G. Restrepo S. 2
David Reyes-Guerra 2
Jose H. Rizo 2
Walter D. Rudeen 2
Russell Schofield 2
Ingvar Schousboe 2
E. C. Sword 2
Fernando Vega 2
J. M. Warne 2
Leon A. Yacoubian 2
Roger M. Zimmerman .. 2
Antonio Zuniga Ayala ... 2

New Members

The Board of Direction approved applications in the following categories: 88 Individual, 2 Corporation, 22 Junior, and 16 Student, making a total of 128 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,260.

INDIVIDUAL

ANTHOPOULOS, PHILOCLES, Philadelphia, Pa. (Cons. Struct. Engr.)
ARMSTRONG, W. GENE, Columbia, Mo. (Constr. Supt., Buckley Constr. Co.)
BAKER, RAYMOND S., La Vega, Dominican Republic (Asst. Supt., Compania Constructora Elmhurst C por A)
BAMPTON, NORMAN, Hartford, Conn. (Chf. Engr., Henry Souther Engrg. Co.)
BEATON, W. H., Pointe Claire, Que., Canada (Part., Beauchemin-Beaton-LaPointe, Cons. Engrs.)
BIRD, JOSEPH, Salisbury, Southern Rhodesia (Struct. Engr., L. S. C. Brunette & Partners)
BLANK, CESAR A., Caracas, Venezuelas
BLASCHKE, E. H., Houston, Texas (Staff Engr., Brown & Root, Inc.)
CARLSON, C. H., Des Moines, Iowa (Pres., Carlson Rockey, Inc.)
CASSIS, E. GEORGE, Hillsboro, Ill. (Part., Knostman & Cassis)
CHARLONS GARCIA, JULIO IGNACIO, Matanzas, Cuba (C.E., Ministerio de Obras Publicas)
CLEMENTE, ANGEL ESCALERA, Mayaguez, P. R. (Surv.)
CORE, HARVEY, Baton Rouge, La. (Contr.)
CROOKS, R. M., Castro Valley, Calif. (Constr. Insp., Whisman School Dist.)
CRUMP, JAMES R., Youngstown, Ohio (Vice-Pres. & Gen. Mgr., Wickliffe Concrete & Supply)
CRUMPACKER, HAROLD C., Valparaiso, Ind. (Struct. Engr., Pioneer Ser. & Engrg. Co.)
DANAHER, MARTIN DENIS, Wellington, New Zealand (C. E.)
DATTA, S. K., Richmond, Surrey, England (Sr. Engr., Pierhead Ltd.)
DOMENECH, ENRIQUE, Bayamon, P. R. (Struct. Engr.)
DUFUR, W. L., Sioux Falls, S. D. (Constr. Supv., Turner Constr. Co.)
DWARAKAPRASAD, VEERAPPA, Boulder, Colo. (Asst. Engr., Mysore Pub. Wks. Dept. (India)
EEFTING, W. T., Miami, Fla. (Dir. of Engrg., City of Miami)
ELLIOTT, GEORGE S., Vancouver, B. C., Canada (Gen Mgr., Constr. Est. & Surv. Ser. Ltd.)
ERDEI, CHARLES, London, England (Design Engr., Ove Arup & Parts.)
FONDRIEST, F. FREDERICK, Columbus, Ohio (Proj. Engr., Battelle Memorial Institute)
FORKER, GEORGE, New Hyde Park, N. Y. (Gen. Mgr., Precast Bldg. Sects., Inc.)
FUCHSSTEINER, W., Darmstadt, Germany (Cons. Engr.)
GREEN, JOHN F., Danbury, Conn. (P. E. & Land Surv.)
HATCHER, DAVID S., Urbana, Ill. (Rsch. Asst., Univ. of Ill.)
HAUSMANN D. A., South Gate Calif. (Rsch. Dir., Am. Pipe & Constr. Co.)
HENNING, GEORGE S., San Mateo, Calif. (Emsco of San Francisco)
HODSON, G. B. G., Salisbury, Southern Rhodesia (Struct. Engr., L. S. C. Brunette & Parts.)
HOLMES, WILLIAM H., Syracuse, N. Y. (Pres., Holmes Bros., Inc.)
HOUK, NORRIS A., Santa Cruz, Calif. (Cons. Engr.)
ILARIA, JOSEPH E., Wharton, N. J. (Rsch. Chem., Lock Joint Pipe Co.)
IRANI, SHERIAR NOSHERWAN, New York, N. Y. Engr., Ammann & Whitney)
KIMBALL, L. ROBERT, Ebensburg, Pa. (Cons. Engr.)
LAWRENCE, WM. H., Westville, N. J. (Engr., John D. Lawrence, Inc.)
LEWIS, JOHN R., FPO San Francisco, Calif. (Deputy Area Engr., Kwajalein Area, U. S. Army Corps of Engrs.)
MAGNUSSON, KJELL, Hohog (near Malmo), Sweden
MALANKA, CARMINE F., Denville, N. J. (Est., C. F. Malanka & Sons, Inc.)
MANKIN, W. H., Portland, Ore. (Chf. Engr., Donald M. Drake Co.)
MANSON, JAMES JOHN, Detroit, Mich. (Vice-Pres., Nat. Conc. Prods.)
MARTIN, ALLEN R., Dutch John, Utah (Head, Contr. Sect., USBR)
MARTIN, DONALD A., Tacoma, Wash. (Gen. Mgr., Meade Sand & Gravel Co.)
MCKILLIP, R. L., San Francisco, Calif. (Sales Engr., Columbia-Geneva Steel Div., U. S. Steel Corp.)
MORGAN, JAMES F., JR., Bowmansville, N. Y. Quality Control Engr., Buffalo Crushed Stone Corp.)
NELSON, B. HAROLD, Wallingford, Conn. (Tech. Sales, Sika Chem. Corp.)
NICKEL, DONAVON D., Westminster, Colo. (Struct. Engr.)
NORA, JOHN, JR., Palm Beach, Fla. (Part., Constr. Supt., Nora Constr. Co.)
OPPERMAN, A. PETERS, Ann Arbor, Mich. (Instr., Univ. of Mich.)
PATEL, KANTI, Bristol, England (Struct. Engr.)

PIETROWSKI, FRANK, JR., New York, N. Y. (Sr. Engr., Arabian-Am. Oil Co.)
PLOTKIN, HAROLD, Harrisburg, Pa. (Field Engr.)
POWER, C. A., Wellington, New Zealand (Part., Orchiston, Power & Assocs.)
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QUIRT, F. H., Winnipeg, Man., Canada (Prod. Mgr., Pressur-Crete Ltd.)
RAY, J. B., JR., Tulsa, Okla. (Exec. Sec., N. E. Okla. Ready-Mix Conc. Assn.)
REDDI, E. S., Boulder, Colo.
RICHARDS, OWEN, Kensington, Md. (Mfr's Rep., Perma-Rock Prods., Inc.)
RICHMOND, MORRIS F., Pittsburgh, Pa. (Sr. Engr., Dravo Corp.)
ROBERTS, KENNETH L., Fairfax, Va. (C. E.)
RODRIGUEZ, JUAN F., Mayaguez, P. R. (Instr., Gen. Engrg. Dept., Univ. of P. R.)
ROFAIL, ABDO ESHAK, Cairo, Egypt (Major, Engr. Corps)
ROGERS, PERCY L., Riverton, Va. (Prod. Mgr., Riverton Lime & Stone Co.)
RUFF, ROBERT C., Nagoya, Japan, Erik Floor & Assoc., Inc.)
SAFIE, CARLOS EDUARDO, Washington, D. C. (Struct. Engr.)
SAVAGE, RICHARD L., Arlington, Mass. (Asst. Prof., C. E., Tufts Univ.)
SCHLECHT, JOHN A., Williston Park, N. Y. (Tech., Etna Pozzolana Corp.)
SCHWEIGER, ROBERT T., Kansas City, Mo. (Engr., Supt., Est., Schweiger Constr. Co.)
SEELER, GERHARD, Stamford, Conn. (Struct. Engr., Werner, Jensen & Korst)
SERAPHIDES, ALEXANDER G., Pittsburgh, Pa. (Struct. Designer, Houser & Carafas Engrg. Co.)
SHIVELY, ROBERT GILBERT, Cleveland, Ohio (Instr., Struct. Engr., Kent State Univ.)
SHOEMAKER, EDWYN L., JR., Willow Grove, Pa. (Quality Control, Atlantic Prestressed Conc. Co.)
SIPOS, ANDREAS, Philadelphia, Pa. (Struct. Engr., Allied Chem. Corp.)
SNOWDEN, WALTER L., Austin, Texas (Sales Engr.)
SOMASEKHARA, PARAMANAHALLI VENKATASWAMYGOWDA, Bangalore, India (Chf. Engr. Gen., Pub. Wks. Dept., Govt. of Mysore)
SZOSTAK, BOLESLAW, Evansville, Ind. (Struct. Designer, John R. Wilkie, Cons. Engr.)
TAYLOR, FORREST S., Friant, Calif. (Chf. Engr., Owner, Calif. Indust. Minerals Co.)
THOMPSON, JAMES L., Tallahassee, Fla. (Plant Mgr.)
THON, JOSEPH, Hamilton, Ont., Canada (Proj. Engr., City of Hamilton)
TRUSS, MONTGOMERY V., Birmingham, Ala. (Mgr., Engrg. Ser., Vulcan Matls. Co.)
WARDER, S. J., Saskatoon, Sask., Canada (Struct. Engr., Underwood, McLellan & Assocs., Ltd.)
WILSON, J. D., Rialto, Calif. (Plant Mgr., Conc. Conduit Co.)
WINTERKORN, HANS F., Princeton, N. J. (Rsch. Instr., Princeton Univ.)
WORTHINGTON, J. D., San Francisco, Calif. (Chf. C. E., Pacific Gas & Elec. Co.)
YOON, CHANG SUP, Seoul, Korea (Instr., Seoul Nat. Univ.)

CORPORATION

CARL N. SWENSON CO., INC., San Jose, Calif. (C. W. Swenson, Pres.)
WIGTON-ABBOTT CORP., Plainfield, N. J. (Irwin Borsuk, Vice-Pres.)

JUNIOR

ALONSO-ALVARADO, RICARDO, Bogota, Colombia (C. E., Spinel & Cia. Ltda.)
ARAMBURO B., LUIS, Popayan, Colombia (Prof. of Design & Analysis of Structs., Univ. del Cauca)
ARANGUREN, BENITO, Caracas, Venezuela (OFITEC C. A.)
BASLER, H. G., Edmonton, Alta., Canada (Sessional Instr., Univ. of Alta.)
BIERSTOCK, DON, Winnipeg, Man., Canada (Engr.)
CAPO, JOSE RAFAEL, Mayaguez, P. R. (Instr., C. E., College of Agric. & M. A.)
ENGEL, GUENTHER W., Troy, N. Y. (Struct. Engr., Sommers, Munninger & Molke)
FERNANDEZ, JOSE MANUEL, Mexico, D.F., Mexico (Chf. Engr., PRESCON of Mexico)
GLOPPEN, EDWARD C., La Grange, Ill. (Engr., Am. Marietta Co.)
HOWARD, ROBERT S., Southfield, Mich. (Dev. Engr., Detroit Edison Co.)
JAMES, THOMAS M., New York, N. Y. (Struct. Engr.)
JOHNSON, LAURANCE P., Houston, Texas (Struct. Engr., Brown & Root, Inc.)
MEYERS, BERNARD L., Columbia, Mo. (Instr. & Proj. Engr., Univ. of Mo.)
NASSER, ANDREW R., Los Angeles, Calif. (Struct. Designer, Johnson & Nielsen)
NOUBLEAU, VICTOR, San Salvador, El Salvador (Design Engr., Comision Ejecutiva del Puerto de Acajutla)
PEACOCK, W., Edmonton, Alta., Canada (Struct. Engr., Dept. of Pub. Wks., Govt. of Alta.)
PULIDO M., ILLA, Caracas, Venezuela
REDMOND, JAMES F., Minneapolis, Minn. (Struct. Engr., Thorshor & Cerny)
RICHARDSON, DON R., Agincourt, Ont., Canada (Struct. Engr., W. O. Chisholm, Cons. Engr.)
SAAVEDRA, SANTIAGO, Bogota, Colombia (C. E.)
SALAM, ABDUL, Karachi Airport, West Pakistan (Arch. Draftsman, Pakistan International Airlines)
TAYLOR, D. J., Edmonton, Alta., Canada (Plant Engr., Edcon Block Ltd.)

PINTURAS PARA HORMIGON

Spanish translation of two ACI reports on the application of paint to concrete. Prepared from ACI Standard 616-49, **"The Application of Portland Cement Paint to Concrete Surfaces,"** and the Committee 616 report on **"Guide for Painting Concrete."***

●

$1 per copy

●

ACI PUBLICATIONS DEPARTMENT

*Both reports are available as separate booklets in English.

STUDENT

BATRES SANTOLINO, ARTURO, Guatemala City, Guatemala (Univ. de San Carlos)
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BRAUER, TILL, Rio Piedras, P. R. (College of Agric. & Mech. Arts)
CORNELL, DWIGHT M., Westerly, R. I. (Worcester Poly. Inst.)
DEL VALLE, JUAN M., Havana, Cuba (Univ. of Havana)
DIAZ GARCIA, HUGO, Bogota, Colombia (Univ. Javeriana)
DI ZOGLIO, HARRY FRANCIS, Johnston, R. I. (Worcester Poly. Inst.)
GOUDREAU, JERRY, Cleveland, Ohio (Univ. of Notre Dame)
GRIGG, CHARLES D., Champaign, Ill. (Univ. of Ill.)
HINDO, KHALID RAPHAEL, Notre Dame, Ind. (Univ. of Notre Dame)
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MONTES DE OCA, AUGUSTO CESAR, Barahona, Dominican Republic
ORTEGA, GERARDO, Havana, Cuba (Univ. de Havana)
OTERO, MANUEL GERGUIN, Austin, Texas (Univ. of Texas)
PANANANDA, A., Worcester, Mass. (Worcester Poly. Inst.)
STEPHEN, ROY M., Berkeley, Calif. (Univ. of Calif.)

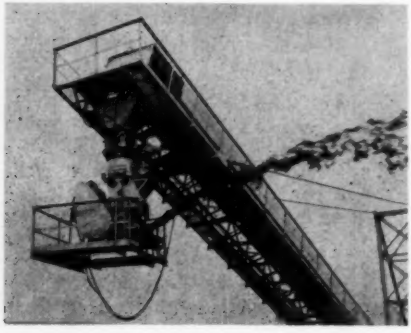
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Swivel thrower

When handling sand, the Swivel-Piler increases conveyor storage range up to ten times, according to the developer, Stephens-Adamson Mfg. Co.

The unit, consisting of a centrifugal thrower unit complete with hopper and swivel-joint, can be mounted as a unit beneath the discharge end of practically any portable or fixed belt conveyor. The unit will throw material in any direction over an arc of 270 deg. The thrower features a short, endless high speed conveyor belt which operates on centrifugal action principle. The trajectory of material can be raised or lowered from 8 to 40 deg by means of an operating adjusting lever.

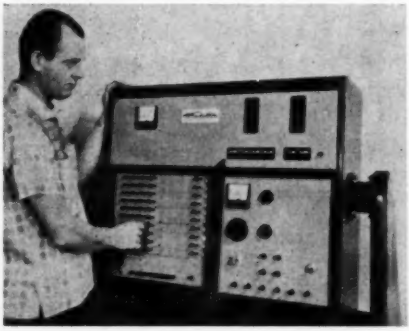


The unit is available in 10 and 16 in. wide belt sizes.—Stephens-Adamson Mfg. Co., Ridgeway Ave., Aurora, Ill.

Batching console

The Selectron, a compact batching console, features a simple system for presetting multiple formulas and operating batching plant remotely.

Up to ten different formulas of ten different materials can be preset in advance. Additional formulas in increments of ten are optional. Selection of formula or change from one formula to another is immediate. Individual remote dials permit reading of up to eight corresponding dial scales on batch deck, enabling the console to be positioned remotely in yard office, at mixer truck charging station, or at any other desired location.



The electronic components of the console are convenient subassemblies, easily interchangeable, according to manufacturer. The console can be installed on existing concrete batching plants and maintained by the ready-mixed concrete producer or contractor's own personnel. Standard unit is equipped with full batch-start and batch-dump interlocks.—Noble Co., 1860 7th St., Oakland, Calif.

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